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A TREATISE ON THE

PRINCIPLES AND PRACTICE

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PREFACE.

Maritime Engineering is a science of peculiar and vital importance to the national and commercial welfare of an insular people.

The subject, however, in its entirety, is much too extensive to be dealt with within the limits of a single volume, and, even in treating that section of it relating to docks, the author feels that he is but touching on the fringe of a theme fraught with manifold possibilities and capable of great future development.

His aim throughout has been to deal thoroughly rather than extensively, and to investigate in detail rather than in general, leaving nothing undone, in order that premises and conclusions alike might be presented in their completest and most intelligible form. And here it may be remarked that, while the book has been written largely, and even mainly, for the student, it is hoped that it will not be without some value for reference purposes to the expert and, indeed, to all who are in any way concerned with this branch of engineering and its cognate interests.

The compilation of such a work has naturally entailed, in addition to a basis of long personal experience, much correspondence and research, and the author takes this opportunity of acknowledging his indebtedness to many professional friends, who have contributed valuable information and who have otherwise rendered him assistance in a task of no inconsiderable difficulty. To the Councils of the Institution of Civil Engineers, the Institution of Mechanical Engineers, the Institution of Naval Architects, the American Society of Civil Engineers, and the Liverpool Engineering Society, he tenders his thanks for permission to reproduce diagrams and to make extracts from papers published in their respective

Minutes of Proceedings, as well as to the writers of the papers for their personal sanction. In addition to these gentlemen, the author feels that he cannot omit to specify his great obligation to Mr. A. G. Lyster, Engineer-in-Chief to the Mersey Docks and Harbour Board, for the privilege of making use of much valuable material; to M. Pastakoff, of St. Petersburg; M. Delachanal, of Havre; and to many other English and Continental engineers for esteemed contributions relating to recent practice at various ports.

The Editors of *Engineering* and of *The Engineer* are thanked for permission, very courteously accorded, to make extracts from the columns of their journals. A number of well-known engineering firms have also kindly placed at the author's disposal diagrams of plant and appliances manufactured by them.

Whilst every care has been taken to ensure the accuracy of statistics and calculations, it is possible that a few errors may have crept in and escaped detection. It is trusted that these, if discovered, may prove to be of minor importance; but, in any case, the author will be very grateful for an intimation of them.

BRYSSON CUNNINGHAM.

LIVERPOOL, *January*, 1904.

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DOCK ENGINEERING.

CHAPTER I.

HISTORICAL AND DISCURSIVE.

INTRODUCTORY DEFINITIONS—PORTS AND THEIR FUNCTIONS—THE DEVELOPMENT OF MARITIME ENGINEERING—THE FIRST WET DOCK—THE HOWLAND GREAT WET DOCK—RECENT PROGRESS—DOCK ADMINISTRATION—HISTORICAL NOTICES OF THE PORTS OF LONDON, LIVERPOOL, NEW YORK, GLASGOW, HAMBURG, ANTWERP, MARSEILLES, ROTTERDAM, CARDIFF, AND OF THE TYNE PORTS.

Introductory Definitions.—In the terminology of maritime engineering, a Dock is an artificial repository for shipping.

This definition, admittedly vague, and at first sight unsatisfactory, not to say incomplete, is, nevertheless, the only one, apparently, which can be devised to cover the manifold and diverse applications of the word. On consideration, it will be seen that its terms do not admit of further restriction.

Docks are divisible into three classes, with widely different characteristics and functions, viz. :—Wet Docks; Dry or Graving, and Slip Docks; and Floating Docks.

Wet Docks are areas of impounded water within which vessels can remain afloat at a uniform level, independent of external tidal action. They have also been termed Floating Docks, in which case the epithet denotes the object for which the dock exists; but as this name is liable to be confused with that in which the epithet is descriptive of the dock itself, it is not at all suitable, and should be avoided.

Dry Docks are those from which water can be temporarily excluded, in order that repairs to the hulls and keels of vessels may be effected. When the vessel is floated into the dock, and the water removed by natural or artificial means, the term *Graving Dock* is appropriate. When the vessel is partially withdrawn from the water by means of ways, the remaining water being excluded as before, the term *Slip Dock* is used.

Floating Docks are frames or structures capable, by reason of their own flotation, of raising ships completely above water, and of maintaining them in that position during the execution of repairs.

The term dock is also applied, though somewhat loosely, to tidal basins—that is, to areas of partially-enclosed water in free communication with

the sea. The functions of basins in many cases coincide with those of docks, so that some elasticity of nomenclature is not without justification. At the same time, it must be affirmed that the term is not correctly applicable to basins, though the distinction will not be too rigidly insisted upon in the course of this work.

Primarily, it appears that a dock was devoted entirely to shipbuilding and ship-repairing purposes. When fitted with appliances for the exclusion of tidal water, it was distinguished as a dry dock; otherwise, it was a wet dock, and the ship was only accessible during periods of low water.

Ports and their Functions.—Though by no means a unique or even an essential feature, a dock-system nevertheless constitutes the most important appendage of a port.

Ports may be regarded from two distinct points of view—either as the termini of great ocean trunk lines of communication, or as intermediate stations on the entire route from the manufactory to the mart, from the producer to the consumer. Each aspect has its own special characteristics and problems, alike interesting to the engineer. As a terminus, the port must be provided with ample accommodation and sheltered berths. As an intermediate station, it must be readily accessible and fully equipped with all necessary appliances for the speedy transfer of merchandise between ship and shore.

The subject of ports as a whole, however, exceeds the scope of the present treatise, for it would involve the discussion, not only of docks, but also of harbours, channels, waterways, and roadsteads.

Natural havens and roadsteads do not fall within our purview at all, neither have we to concern ourselves with those large areas of safe anchorage which are formed and protected by breakwaters. Upon smaller areas, more completely enclosed and designated harbours and basins, we shall touch but lightly. Our immediate purpose is to deal with spaces of moderate extent, more or less continuously cut off from external influences, and properly called docks.

The Development of Maritime Engineering.—While harbours have constituted prominent features in connection with maritime intercourse from the remotest times, docks are a comparatively modern innovation. We should have to go very far back indeed into the history of navigation to trace the origin of artificial harbours. Natural harbours and creeks have, of course, always been available; but their situation and accommodation, even in early days, sometimes proved not altogether satisfactory. Accordingly, we find that the Phœnicians protected their ancient ports of Tyre and Sidon on the Levantine Coast by means of rubble breakwaters. Carthage, the home of their descendants, likewise possessed a harbour enclosed by moles. Rome, the vanquisher of Carthage, has left, despite the ravages of time and disuse, many traces of maritime engineering structures along the coasts of Italy; nor is Greece lacking in striking examples of harbour works upon her classic shores. With the downfall of the Roman Empire commer-

cial enterprise found a home in the still flourishing ports of Venice and Genoa. Later, Spain set her grasp upon ocean trade, investing Barcelona and Cadiz with a glory, some vestiges of which cling to them still. Later again, the phlegmatic Dutch took over the supremacy, and, with patient toil and perseverance, laid the foundations of their ports within the very domain of the sea itself.

In our own country, notwithstanding the spirit of naval adventure which animated the Cabots, the Drakes, the Raleighs, the Frobishers, the Hawkins, and many other heroes of the Tudor period, little was done to improve such facilities as were naturally possessed by towns upon the seaboard. Dover was for a long time perhaps the only port of real note developed in any way by artificial agency. Subsequently Bristol, Plymouth, London, and Leith, amongst others, rose to importance, but most of our present leading ports are of quite recent growth. Liverpool, Hull, Glasgow, and Newcastle afforded very trifling accommodation for shipping a century ago. Cardiff, Barrow, and Middlesbrough have existed as ports for little more than fifty years. Twenty years ago Barry was unknown, and Manchester an inland town.

The First Wet Dock.—The distinction of having created the first wet dock has been the subject of some discussion and the cause of not a little rivalry between the ports of London and Liverpool. According to such evidence as is forthcoming—and some of it is conflicting and inconclusive to a degree—the balance appears to incline in favour of the former place. As regards Liverpool, it is generally admitted that parliamentary authority was obtained for the construction of a wet dock in the year 1708, during the reign of Queen Anne. This dock was built and opened, apparently, very shortly afterwards, the engineer being Thomas Steers. But, according to the "City Annals" appended to Gore's Directory of Liverpool, the dock was already in existence in 1700, and a date (June 8) is given on which the first ship, the "Marlborough," entered it. Possibly these conflicting statements are reconcilable if we regard the earlier dock as having been of the nature of a tidal basin, which was afterwards converted into a wet dock by the addition of entrance gates.* Some interest attaches to this "Old Dock" as it is termed. It was four acres in extent, and was designed to afford accommodation for 100 vessels, and so arranged as to have not less than 10 feet of water within it at low neap tides, with a sufficiency at spring tides to take the smaller class of warships. The dock no longer exists except in name, although the level of its sill still supplies the zero or datum in vogue throughout the Mersey Dock Estate in preference to the, elsewhere, more generally accepted Ordnance Datum.

On the other hand, as regards London, the inception of the Surrey Commercial Docks is said to date from 1660 or 1666; at any rate, there is decisive evidence that an Act of Parliament for the construction of a wet dock at Rotherhithe received the Royal Assent in 1696. The date when

* This is mere conjecture, and a dubious solution at the best.

the dock was opened is not recorded, but that it was in use in the year 1703 is testified by an old description (undated) of the dock which, with an engraving, is retained in the Board-room of the Surrey Commercial Dock Company. Being not without interest as an old-world document, throwing light upon conditions which prevailed two centuries ago in regard to the management of ships in port, a copy of it is reproduced here.* The original name of the dock, "The Howland Great Wet Dock," has since been replaced by that of the "Greenland Dock." Mrs. Elizabeth Howland was the wife of John Howland, of Streatham, Surrey, and the mother-in-law of the Marquis of Tavistock, afterwards second Duke of Bedford.

HOWLAND GREAT WET DOCK,

In the Pariſh of Rotherhithe, or Redriff, belonging to Mrs. Howland, of Streatham.

This dock hath been found a very ſafe repoſitory for ſhips, which was fully proved in that terrible and violent ſtorm which happened on the 27th November, 1703, when by the extremity of the wind all the ſhips in the river, which rode either at chains or their own moorings, were forc'd adrift, and confuſedly driven on the North ſhore, where ſome were left, and moſt received great damage. Then, of all the ſeveral ſhips depoſited in this wet dock there was only one injur'd, and ſhe only in her bowſprit, which was in a great meaſure imputed to too ſecure a negligence in the perſons who moor'd her there. This may remain a laſting evidence of the great ſervice ſuch a repoſitory for ſhipping is to our navigation; eſpecially if it be conſidered that this fatal ſtorm happen'd ſoon after the planting of thoſe trees, which are on the ſouth and north as a fence to the dock from winds, and which are now grown to a conſiderable bulk; and alſo before that range of houſes were built to the weſt, and the pailings ſet up to the eaſt, and on each ſide; ſo that now, in the hardeſt gales of wind that have within theſe late years happened, notwithſtanding the large extent of the water, the wind does not give any ſuch motion to it, as can endanger tho ſmalleſt boat in paſſing it any way over, and tho' very deep loaded. And as ſhips are here ſo well ſecur'd from any ſtorm that may happen, ſo they are entirely defended from the hazard and damage which accrueth to them often in the river, by hard froſts. For by the driving of the ice in the river, if they ſhould continue in the ſtream on float, their cables would be cut; to prevent which and to preſerve their bottom, they are forc'd to take up with ſhore births, which often are ſtraining and uneaſy to the ſhips, and require a conſtant care and charge to preſerve them, by ſhoring or ſhifting, as it may happen, by the ice's driving under them. And notwithſtanding all the care which can be taken, the bottoms of

* Vide *Min. Proc. Inst. C.E.*, vol. c., p. 93.

ſhips are ſo raked by the ice, that it is often a conſiderable addition in the charge of refitting, if no other more material damage happens to them thereby. Whereas the ſhips here depoſited, lye always water borne, without the leaſt rubbing of the ice, or any further care or charge for their preſervation, as fully appeared by the laſt great froſt in 1715. *Ships are likewiſe here more effectually ſecur'd from the peril of fire; there being proper cook rooms provided on ſhore, and no fire ſuffered to be on board. But if neither ſtorms, nor ice, nor fire, be conſidered, ſhips are here depoſited at a much leſs charge and a much greater ſecurity than in the river; which any one may eaſily evince, if he will calculate the wearing their cables or the charge of the chain, the frequent ſhifting of the moorings, and other neceſſary incidents, which do and will happen in the river, and compare them with the moderate rates wet-docking is by this work reduced to.*

Deſcription of the Dock.—*The outward gates of the wet-dock, leading to the Thames,* 21 *foot high, and* 44 *foot wide, open'd to let in the ſhip.*

The baſon, or gut, leading to the great wet-dock, 44 *foot wide,* 150 *foot long.*

The inward gates, of the ſame height and breadth with the outward, but ſtronger, by reaſon they bear the great weight of water in the dock, which ſometimes flows within a foot of the top of theſe gates, and is kept pent up within 4 *foot thereof.*

The great wet-dock, wherein at good ſpring tides there is ſeventeen foot of water, over the cell againſt which the bottom of the gates ſhut; ſo that it would commodiouſly receive his Majeſty's third-rate ſhips.

The dimenſions of the dock are from eaſt to weſt 1,070 *feet; from north to ſouth, at the weſt end,* 450 *feet, and from north to ſouth, at the eaſt end,* 500 *feet; ſo that it would contain upwards of* 120 *ſail of the largeſt merchant ſhips, without the trouble of ſhifting, mooring, or unmooring any in the dock, for taking in or out any other.*

This dock when full at a ſpring tide, contains, by a moderate computation of 40 *foot ſolid to the ton,* 228,712 *tons of water, being much larger than the famous baſon of Dunkirk, or any pent water in the world.*

The maſt crain, for taking out and ſetting in maſts in ſhips in the wet-dock, which anſwers the end of an hulk, with proper pits and crab for careening three or four ſhips at once.

Recent Progress.—To whichever of the two rival ports the honour be allocated (and this is a matter of no great moment), at any rate it is apparent that the first English dock dates back no further than the commencement of the 18th century. But what perhaps is more remarkable still, is that during the next hundred years, despite the enormous increase in oversea trade and the great development of lines of inland navigation, no works of any note were undertaken for the extension or improvement of dock accommodation.

It is very difficult to realise that, up to the last decade of the 18th century, the Thames only possessed its one dock (and that devoted to the whaling trade), while Liverpool had but three, and these of inconsiderable extent.

It was left to the 19th century to witness a great revival in dock and harbour engineering. Great forces which had been slowly gathering throughout the Georgian period eventually came to a head. The sudden growth of commerce consequent upon the advent of steam power, the expansion of the empire and the opening up of virgin territory, gave an impetus to national policy which resulted in the adoption everywhere of vigorous and energetic measures. The history of the Victorian era is a long and triumphant record of feats of maritime engineering skill adorned by the names, amongst others, of Rennie, Smeaton, Stevenson, Hawkshaw, Messent, Coode, Hartley, and Lyster, and attested by the splendid array of docks and harbours which line the English coast to-day.

Nor is there any sign yet of a diminution in the activity which has produced such magnificent results. Fresh undertakings are demanded daily to correspond with each succeeding development of naval architecture and with each access of national prosperity. From the point of view of national vitality this is, indeed, no time for relaxation of effort. Powerful trade competitors have arisen in nations who, admittedly outdistanced before, now openly dispute the British claim to the sovereignty of the seas. Renewed exertions will have to be made, both to retain trade and to cope with its altered conditions. Hence the necessity, on the part of port authorities, for a watchful and attentive attitude, ready to note each impending change and its probable consequences; to seize each favourable opportunity for fresh enterprise, and by decision and energy to utilise it to the fullest extent. Only in this way can ports, as well as nations, hold their own.

Dock Administration.—Docks are to be found under five different systems of management, and though the question of administration is one of economics rather than of mechanical science, it merits at least a passing reference. The five systems of administration may be enumerated as follows:—

(1) *Private or Public Companies,* ad hoc.
(2) *Railway Companies.*
(3) *Municipalities.*
(4) *Public Trusts.*
(5) *Government Departments.*

Of these it may be said that *private companies* are in the least favourable position for maintaining their docks in an efficient condition, or for meeting the needs of a growing port. Dock engineering works are particularly costly, and the return on capital thus invested, except in rare instances, will not bear favourable comparison with dividends arising from securer sources.

Hence there must inevitably be undue economy and even parsimony in management, and a reluctance to undertake fresh expenditure on works, however beneficial or necessary.

Railway Companies derive a considerable amount of indirect benefit by the proprietorship of docks in touch with their respective systems, quite apart from any specific receipts locally. The facilities for the direct transfer of goods from rail to ship, and *vice versâ*, are greatly increased without any corresponding augmentation of staff and without friction of negotiation. The diversion of traffic to their lines is often sufficient to compensate a company for the otherwise unremunerative working of their docks.

Municipal Councils, nominally the controlling authorities, generally delegate their powers of dock management to a sub-committee, with results that have not been uniformly successful. Town Councillors are elected on a variety of grounds, sometimes personal, but mainly political, and often without the remotest bearing on shipping matters. Now, however versed in the direction of purely urban affairs a councillor may be, it is obvious that, without some active participation in maritime affairs, he will lack the requisite technical knowledge to enable him to deal satisfactorily with important questions affecting the mercantile marine. Hence in such a committee the likelihood of uncertain counsels, sometimes unduly timorous, sometimes the reverse.

Public Trusts, specially elected from the classes most intimately associated with the use and exploitation of docks, constitute perhaps the most satisfactory of all forms of government. On a body of this kind would be proper representatives, chosen by an electorate of shipbuilders, shipowners, merchants, and traders; of all, in fact, who were connected with the shipment of goods, the qualification being the payment of dock or port dues. The particular knowledge possessed by such a body would be, and is eminently calculated, to develop the efficiency and prosperity of a port, the efforts of the members being stimulated by a certain amount of self-interest. It must not be overlooked that the welfare of the port involves the welfare of the town, and that the two suffer or flourish together. Hence the necessity for specialist management in both cases.

Control by a Government Department, which would naturally involve the inclusion of all ports within one national jurisdiction, cannot be considered a desideratum. Speaking generally, it is admitted that there is a lack of initiative and a diffusion of authority in governmental departments which are not adapted to the successful carrying on of commercial undertakings. The almost inevitable result of this system would be the stifling of private enterprise, and the abandonment of that local patriotism which constitutes the best guarantee of the vitality and energy of a port, at the same time that it affords the best augury for its continued prosperity.

We now pass on to a brief *résumé* of the more prominent historical facts connected with the development of some of the most important ports of the world. It would be difficult to assign to them any satisfactory order of

precedence. Navigation returns fluctuate considerably, and with them the relative positions of the ports concerned. No attempt, then, will be made to preserve any particular sequence except that attaching to general prominence and representative character.

The Port of London.

The Port of London has long maintained an indubitable supremacy. At the beginning of last century, however, it received no more than 4,000 ships annually, of which number more than half were coasting vessels, and the aggregate tonnage scarcely exceeded half a million. In 1901 the number of ships which entered and cleared the port was 53,230, and the tonnage 31,157,015.

The Greenland (or Howland) Dock, with its area of 12 acres and quayage under a mile, held its unique position until the year 1790, when the Brunswick Dock was constructed by a shipbuilder on the site of the present West India Dock. The shipping at this time was mainly accommodated at "legal wharves" at the river side or at moorings amidstream. The delay which arose in this way from stoppages of the navigable channel and the enormous losses sustained by robberies, created a scandal of such moment that the Government of the day was obliged to take action, and parliamentary powers were obtained for the redemption of some of these legal wharves by compensating their owners. At the same time an Act was passed authorising the construction of the West India Dock. This dock was so named from its appropriation to the West Indian trade, and all vessels engaged in that trade were compelled to use the dock, which had the monopoly conferred upon it for twenty-one years. The date of opening was 1802. It was followed in 1805 by the London Dock, which was endowed with a monopoly of vessels engaged in the conveyance of wine, spirits, and tobacco. The East India Dock was opened on equally protective lines in 1806. The first free dock (St. Katharine's) did not come into existence until the years 1827-28. After this a long interval elapsed, until the construction of the Royal Victoria Dock in 1855. This dock, situated nearly opposite Woolwich, is a very important one. Its length is 3,000 feet and its width 1,050 feet; and, with its appurtenances, it added 90 acres to the water area of the port.

The Millwall Docks—in reality but one, shaped like the letter L—were next built in 1868. They have a water area of 35 acres. In 1870 came the opening of the South-West India Dock, parallel to the other two India Docks; like them, stretching across the Isle of Dogs, and having a river connection at each end.

In 1880 another large dock, the Royal Albert, added very materially to the extent of the port. With its entrance basin it has an area of 84 acres. It is in close connection with the Victoria Dock, being joined to it by a channel.

The available space in the higher reaches of the river was now becoming very restricted, and, moreover, the congestion of traffic caused much interference with, and even prevented, any rapidity of navigation. Accordingly, the next group of docks, the TILBURY DOCKS, were built lower down the river, opposite Gravesend. They consist of a main dock with three parallel branches, in addition to a tidal basin, entrance locks, and graving docks. By this group, opened in 1886, the port was enlarged by 57¼ acres.

The water area of the port now amounts to about 570 acres, exclusive of shallow timber ponds, and it is being added to by important improvements at the SURREY COMMERCIAL DOCKS. These docks, which are situated on the south side of the river, consist of two groups—the Commercial Docks, dating back to the Howland Dock, reconstructed in 1807, and the Surrey Docks, opened in 1812. They are mainly used for cargoes of timber and grain.

The present position of London as a port cannot, however, be regarded as satisfactory. The navigation of the river is impeded by tortuous channels beset with shallows, while trade is hampered by insufficient dock accommodation and diversity of management. The docks in London are the property of several distinct companies, with conflicting interests and independent jurisdictions. They are under the necessity of paying dividends, and their capital is insufficient to meet the growing demands made upon it. The amount of interest earned can only be described as meagre, so that there is little inducement to find additional capital for investments of so comparatively unremunerative a nature. Yet, without this expenditure the docks must rapidly pass into a state of inefficiency and disuse.

How to provide funds for the purpose is a delicate and difficult question. Shipowners complain that port charges and dues are already excessive, while other sources of revenue are not available. Radical constitutional changes are impending, including the formation of a Port Trust, with the absorption of all interests in one body. This will undoubtedly lead to considerable economy in management, and a solution of the financial difficulty will, no doubt, be forthcoming. The matter has little interest from an engineering point of view, and concerns but indirectly the province of the dock engineer. Hence, we may with advantage leave so thorny a topic for debate in other and more appropriate quarters.

THE PORT OF LIVERPOOL.

The second port in the kingdom, has a history dating back to the year 1338, when it was first made an independent port. Up to the beginning of the 19th century, however, the docks, for which it is now famous, did not cover a greater area than 18 acres, nor in 1816 were there more than 34 acres; but in 1846 the water space had increased to 108 acres, and in 1857, after the inclusion of the Birkenhead docks, to 209 acres, until at length, in 1901, the combined system comprised no less than 558 acres, with a quayage

of 35 miles, the latter being equivalent to two-thirds of that of the quayage of all other wet docks in the world, excluding British ports.

The docks constructed during the latter half of the 18th century were the SALTHOUSE, the GEORGE'S, the KING'S, and the QUEEN'S. These were devoid of quays, and much time and labour were wasted in the transfer and cartage of goods. The PRINCES DOCK was opened in 1821, and five years later the Old Dock was closed. The CLARENCE DOCK was built in 1830, the WATERLOO in 1834, and the VICTORIA and TRAFALGAR DOCKS in 1836. These earlier docks were of very small size, rarely exceeding 10 acres. The CANADA and HUSKISSON DOCKS, constructed between 1850 and 1860, marked a decided advance in this respect, and the size was still further increased in the case of the LANGTON and ALEXANDRA DOCKS, opened in 1881, the former of which contains 21 acres and the latter, 44½ acres. Larger, again, than these are the EAST and WEST FLOATS, on the Cheshire side of the river, containing 59¾ acres and 52 acres respectively; but none of the docks in the Mersey Estate approach the size of the Victoria and Albert Docks at London.

An immense floating landing stage, built in 1847, forms a prominent feature of the river frontage. It was burned down in 1874, but afterwards restored. There are similar, but smaller, floating stages at Woodside and Wallasey.

The tonnage of vessels entering and leaving the port, which in 1831 only amounted to 1½ millions, had nearly reached 19 millions before the end of the century, with a total of about 40,000 vessels. For the year just closed (Midsummer, 1903) the tonnage exceeded 23½ millions.

The management of the dock system, which is perhaps the finest under single control in the world, passed from the hands of a committee of the Town Council in 1858 into those of a public Trust, created by Act of Parliament, and called the Mersey Docks and Harbour Board, which, since that time, has administered it with striking success.

THE PORT OF NEW YORK.

The premier city and port of the United States is possibly somewhat lacking in attraction for the dock engineer in that it has no docks, in the strict sense of the word. What are, by courtesy, termed docks are open areas of water formed by the projection of numerous timber jetties from the face line of the river quays. The city itself lies on an island between the Hudson and East rivers, in a well-sheltered position which calls for no further protection, while at the same time it is close to the open sea. A further reason for the absence of docks is the small range of tide, which does not exceed 5 feet, on an average. The construction of the river wharves, despite some supervision introduced at the beginning of last century, seems to have proceeded on no definite plan or system until the year 1870, when a special department was constituted for that purpose. The city is now

gradually possessing itself of the river frontages, expanding and improving them on systematic lines. In 1870 the length of wharfage was 28 miles; in 1890 it had increased to 37 miles, and since that date it has been considerably augmented.

The Port of Glasgow.

Glasgow is a notable example of a port existing in the face of many natural disabilities. For a long period the Clyde, afflicted with the dual evils of shallowness and tortuousness, was little better than a ditch. Goods were despatched by pack-horses a distance of over 30 miles from Glasgow, to be shipped at the ports of Troon and Irvine, on the Ayrshire coast. At one time it was despaired of ever rendering the river navigable, and the inhabitants, in 1668, acquired a plot of land, some 13 acres in extent, near the village of Newark, about 18 miles distant, where they built a harbour and christened it Port-Glasgow.

The colony thrived for a time. It even grew into importance. In 1710 it was the principal Custom House port on the Clyde. In 1762, it became the site of the first graving dock in Scotland, built under the direction of James Watt. In 1812, the famous "Comet"—the pioneer of steam navigation in Europe—was built here. This vessel plied the river for passengers, and it is recorded that it sometimes took seven hours to accomplish the journey from Greenock to Glasgow—a distance of less than 20 miles. The zenith of Port-Glasgow's prosperity was, however, at length reached. The citizens of the parent city never abandoned their efforts to increase the navigability of the river, and by dint of perseverance they succeeded in effecting some improvement. Shipping was naturally attracted to the more important trade centre and the fortunes of Port-Glasgow declined. It is at the present time dependent upon its shipbuilding yards for its existence.

In 1768, John Golborne, of Chester, reported to the Glasgow magistrates that by suitable works it might be possible to obtain a depth of 4, or even 5, feet as far as the town. He was considered over-sanguine by some, but he more than fulfilled his word, the depth actually obtained being 7 feet. In 1799, John Rennie, of London, advocated a system of low rubble training walls, and these were carried out with such success that the navigable depth in 1806 had been increased to 8½ feet on spring tides; but improvement for some time thereafter was slow. Up to 1836 the depth in the harbour had only been increased to 7 or 8 feet at low water, making 12 feet at high water of neap tides, and 15 feet at high water of spring tides.

In 1824, an impetus was given to deepening operations by the introduction of the steam dredger; and, whereas in 1821, the maximum draught of vessels navigating the river was 13½ feet; in 1830, it was 14 feet; in 1870, 21 feet; in 1880, 22 feet; in 1890, 23 feet, and in 1900, 26½ feet. The

present condition and prospects of the river are thus stated by the Engineer, Mr. W. M. Alston *:—

"The deepening and widening of the river is still going on, the constantly increasing draught of vessels demanding more depth, and more depth involving greater widths in order that the banks may stand. Dredging is presently being executed to 20 feet below extreme low water, or 22½ feet below average low water of spring tides, corresponding with about 32½ feet at high water, spring tides, at Port-Glasgow, and 33½ feet at high water, springs, at Glasgow; and with this depth, the bottom widths range from 120 feet at the River Kelvin to 500 feet at Port-Glasgow. Out of the 16 miles of channel between the harbour and Port-Glasgow, about 10 miles—not continuous—may be said to have attained this depth; while in the remaining 6 miles the depth varies from about 19 feet to 22½ feet below average low water of spring tides."

Owing to the comparatively small range of tides—about 11⅓ feet at springs and 9 feet at neaps—it has not been deemed necessary to equip the Glasgow docks with gates. They are, therefore, properly speaking, tidal basins in which the water is free to rise and fall with the tide. The first dock, the KINGSTON, with an area of 5⅓ acres was opened in 1867. The QUEEN'S DOCK was completed in 1880, and added 33¾ acres to the available accommodation. This dock is situated on the north side of the river. Parallel to it on the south side has been constructed, between the years 1892 and 1897, the PRINCE'S DOCK, with an area of 35 acres. These constitute the present extent of the dock accommodation at Glasgow. Developments, however, are in hand, and a dock of 16¾ acres is in course of construction at Clydebank, 6 miles below Glasgow Bridge, for the service of the coal and mineral trades.

The shipping at Glasgow, which in 1810 only registered 24 sailing vessels and 1,956 tons, in 1900 had increased to 1,605 sailing and steam vessels with a tonnage of 1,582,229. The vessels arriving and clearing at Glasgow in 1900 numbered 21,800, with a tonnage of 7,461,417.

THE PORT OF HAMBURG.

The leading port of the Continent conducted all its loading and unloading operations prior to 1866 by means of open barges, the ships being moored to dolphins, arranged in long rows in natural bays, along the banks of the Elbe, or in the river itself. The earliest basins to be constructed were the SANDTHOR (24 acres), and the GRASBROOK (16⅓ acres), and these were brought into existence between the years 1860 and 1870. In 1881, the port joined the Customs Union of the German Empire, and a period of great activity in dock construction commenced. From the year 1884 onwards the port has been extended by the formation of the BEACON BASIN (44 acres), the HANSA

* Alston on "The River Clyde and Harbour of Glasgow," International Engineering Congress, Glasgow, 1901.

Basin (90¼ acres), the India Basin (27 acres), the Petroleum Basin (19⅜ acres), the Moldau Basin (62 acres), the Saale Basin (30 acres), and the Spree Basin (27¼ acres).

Up to the year 1895, the total water area of the free port, including the river, canals, and side basins, amounted to 941 acres, of which the basins for sea-going ships occupied 328 acres, and those for river vessels 136 acres. There were also 14½ miles of deep-water quayage. Since that date a large scheme of dock extension, on the south side of the river, has been in hand, and is now practically completed. It includes a deep-water basin, for sea-going ships, having an area of 55½ acres, and a shallow-water basin, for river craft, having an area of 96½ acres.

In the year 1900, 13,102 vessels entered the port with a tonnage of 7,909,913, and of these, 8,933 were steamships with a tonnage of 7,124,145.

The Port of Antwerp.

Antwerp is of ancient origin and long held one of the most splendid positions in the history of European commerce. But, in the year 1648, the Dutch inflicted upon its prosperity a blow from which it did not recover for many long years. Victorious in their struggle with the Spaniards, who at that time were proprietors of the North Sea littoral, they insisted, in the treaty of Munster, upon the closing of the Scheldt on this side of the sea; in other words, on the destruction of Antwerp as a seaport. It was not until the year 1795 that the unfortunate city regained its freedom by the terms of the treaty of the Hague. Since then Antwerp has made notable strides towards regaining its lost position, and to-day it ranks as the second Continental port.

Early in the 19th century there were only two docks and some river quays in existence. The docks had an area of 21 acres only, and this remained the extent of the enclosed accommodation until the year 1860, when the Kattendyke Dock was opened. Twenty years later it was extended to a total area of 120 acres. The Africa Dock for large transatlantic steamers, and the America Dock for the petroleum trade—making an addition of 50 acres in all—were begun in 1883 and finished in 1886. Apart from these there is a magnificent stretch of over 3 miles of quay frontage to the River Scheldt.

In the year 1900, 11,488 vessels with a tonnage of 6,688,272 entered the port.

The Port of Marseilles.

Up to the year 1889, Marseilles was the principal Continental port. From that date Hamburg assumed the lead, and, in 1894, Antwerp wrested the second place in order of importance from her former superior. Despite these successive misfortunes, Marseilles still retains a high position amongst

European ports. The town is a very ancient one, but the harbour accommodation has only really been developed within the last fifty years. Prior to 1852, there was only the Old Harbour, 67 acres in extent, which, with a small canal and basin, made the total water area 72 acres. In this year, the Joliette Basin, which had been commenced in 1844, was opened, and gave an additional area of 54 acres, or, with its outer harbour, 56 acres. In 1863 followed the Lazaret and Arenc Basins adding 51 acres, and the Railway Basin with 41 acres. The National Basin was completed in 1881, and its 105 acres raised the total accommodation of the port to 325 acres. In 1893 the construction of a new basin called the Pinède Basin was authorised, the works for which are not yet completed. It will add 65½ acres to the sheltered water area.

In 1900 the number of vessels which entered and cleared the port was 17,254, and the tonnage 12,178,245. In 1901 the figures were 16,802 and 12,877,731 respectively.

The Port of Rotterdam.

The port of Rotterdam possessed a small dock at the close of the 16th century. This, called the Herring Basin, is shown upon a plan dated 1599. For the next twenty-five years there was steady progress. A plan, dated 1623, demonstrates the existence at that time of the Leuve Basin, the Wine Basin, and the Shipbuilder's Basin. But for the ensuing two hundred years very little appears to have been done in the direction of increasing the amount of enclosed water space. The Salmon Basin was brought into existence at the commencement of the 18th century. At this date Rotterdam was only accessible to ships drawing less than 11 feet of water.

It was not until 1873 that a further impetus was given to the expansion of the port, when the King's Basin and the Railway Basin were constructed. Between 1874 and 1879 the Inner Basin and the Warehouse Basin were opened. These were followed, in 1885, by the Rhine Basin, in 1894 by the Katendrecht Basin, and, in 1898, by the Park Basin. In the last-named year was commenced the construction of the Meuse Basin, which adds 145 acres to the dock accommodation of a port already possessing 147 acres on the right bank of the Meuse and 162 acres on the left bank, making 454 acres in all, exclusive of river berths and moorings.

The number of vessels which entered Rotterdam during the year 1900 was 7,268, with a tonnage of 6,483,665.

The Port of Cardiff.

The staple export of Cardiff is coal, and its position in reference to the great coalfields of South Wales has caused the rise of the town from a

population of 2,000, at the beginning of the 19th century, to one of 160,000, at the beginning of the 20th. The development of the port is due to the MARQUIS OF BUTE, who, between 1840 and 1850, commenced the construction of the docks known by his name. In 1901 the docks covered an area of 153 acres (including the new ROATH DOCK). The quantity of coal shipped amounted to 8,000,000 tons, and the number of vessels entering the port was 14,695, with a tonnage of 9,290,785.

THE TYNE PORTS.

What Cardiff is to South Wales, the cluster of ports at the mouth of the River Tyne is to the North-East coast of England. From ancient times Newcastle has been a great coaling centre, with North and South Shields and Tynemouth in close competition. The growth of the trade has been remarkable. At the beginning of the 19th century the export amounted to half a million tons; at the end it was over 12,000,000. The docks are of quite recent origin, the NORTHUMBERLAND, TYNE, and ALBERT EDWARD DOCKS, with their combined area of 129 acres, having been brought into existence during the latter half of last century. In 1901 the number of vessels entering the ports was 14,072, with a tonnage of 8,491,535.

The following tables, condensed from information published by the Board of Trade, will afford some means of instituting an interesting comparison of the ports enumerated. It will be noted, however, that the statistics relate to foreign trade only.

TABLE I.—FOREIGN TRADE OF PRINCIPAL PORTS IN THE UNITED KINGDOM.

TONNAGE OF SAILING AND STEAM VESSELS ENTERED AND CLEARED WITH CARGOES AND IN BALLAST IN THE FOREIGN TRADE DURING THE YEARS 1900-1902.

Port.	1900.		1901.		1902.	
	Entrances.	Clearances.	Entrances.	Clearances.	Entrances.	Clearances.
	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.
Belfast, . .	435,980	249,770	447,855	226,168	479,377	149,560
Cardiff, . .	5,132,523	7,636,717	4,953,980	7,783,077	4,688,088	7,868,556
Dover, . .	973,074	964,476	955,472	950,477	986,908	980,984
Glasgow, . .	1,452,023	2,229,574	1,558,301	2,267,589	1,618,663	2,525,554
Hull, . . .	2,666,598	2,274,137	2,460,830	1,964,526	2,514,663	1,965,875
Leith, . .	1,055,291	982,309	1,023,669	922,085	989,914	890,357
Liverpool, . .	6,001,563	5,666,145	6,465,153	6,171,072	6,843,200	6,314,514
London, . .	9,580,854	7,119,673	9,992,753	7,282,892	10,179,023	7,385,085
Southampton, .	1,613,913	1,395,486	1,645,166	1,417,555	1,689,525	1,534,966
Tyne ports, .	3,897,142	4,894,157	3,831,554	4,840,256	3,615,046	4,754,301

TABLE II.—FOREIGN TRADE OF PRINCIPAL FOREIGN AND COLONIAL PORTS.

TONNAGE OF SAILING AND STEAM VESSELS ENTERED AND CLEARED WITH CARGOES AND IN BALLAST IN THE FOREIGN TRADE DURING THE YEARS 1899-1901.

Port.	1899.		1900.		1901.	
	Entrances.	Clearances.	Entrances.	Clearances.	Entrances.	Clearances.
	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.
Antwerp, . .	6,837,801	6,735,656	6,696,370	6,669,712	7,466,463	7,518,292
Buenos Ayres, .	3,302,145	2,969,196	2,789,581	2,505,323	returns not	available.
Genoa, . .	3,990,306	3,679,973	4,313,604	4,119,372	4,503,895	4,309,075
Gibraltar, . .	4,328,859	4,299,678	4,455,083	4,414,654	4,171,350	4,159,272
Hamburg, . .	7,037,294	7,157,576	7,322,476	7,404,112	7,623,098	7,671,938
Hong-Kong, .	6,720,769	6,716,378	7,021,982	7,000,185	7,383,683	7,340,586
Marseilles, .	4,695,168	4,933,946	4,629,599	4,933,945	4,936,095	5,286,640
New York, .	7,707,477	7,496,279	8,176,761	7,843,529	8,679,273	8,118,427
Rotterdam, .	5,956,437	5,828,331	5,970,395	5,762,967	5,950,446	5,733,763
Singapore, . .	4,416,260	4,409,913	4,836,048	4,833,989	5,456,032	5,453,999

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CHAPTER II.

DOCK DESIGN.

NECESSITY FOR DOCKS—RELATIVE ADVANTAGES OF DOCKS AND BASINS—RESTRICTION IN DESIGN—CONSIDERATIONS IN REGARD TO POSITION AND OUTLINE—VARIOUS FORMS—A MODEL DOCK SYSTEM—RATIO OF QUAY SPACE TO WATER AREA—RATIO OF PERIPHERY TO SURFACE—GROUPED DOCKS—INTERNAL DISPOSITIONS—COST OF CONSTRUCTION—FRESH WATER SUPPLY—SHIP DESIGN—TYPICAL DOCK SYSTEMS AT LIVERPOOL AND BIRKENHEAD, BARRY, BUENOS AYRES, TILBURY, GLASGOW, CALCUTTA, HULL, HAMBURG, LONDON, SUNDERLAND, SWANSEA, HAVRE, AND MARSEILLES—STATISTICS OF REPRESENTATIVE DOCKS.

Necessity for Docks.—In the days before steamships were known, when vessels traversing the ocean highways of the world were built entirely of wood, the question of the provision of docks for the accommodation of shipping had not assumed that aspect of importance and urgency which it has since acquired. It was no uncommon occurrence for a vessel to take the ground at the quayside during periods of low water, and this could be done with impunity when hulls were short and stout, and sides thick and strong. In fact, the experience was a recognised incident in the ordinary course of navigation, and we find one of the advantages claimed for the port of Bristol, two centuries ago, was that the harbour afforded a "soft bed, suitable for the grounding of vessels."

In one respect naval architecture has degenerated since those times. Nowadays, the attenuated plating of an ocean steamer, coupled with its enormous length and weight, would inevitably suffer serious strain, if not collapse, under such drastic treatment. Indeed, to such an extent have strength and stiffness been sacrificed to speed, that the foundering of at least one modern craft * is attributed to the fact that the ends of her keel were lifted momentarily upon the crests of two waves, while the central portion spanned the trough between, and, being unsupported, was fractured by the mere weight of the vessel and its internal fittings.

Except then for small fishing craft, deep water berths in the form of harbours, basins, or docks, capable of maintaining shipping continuously afloat, are necessary features of every modern port. River frontage quays may suffice in minor cases in sheltered situations, but, as a rule, the accommodation thus afforded is insufficient.

The question whether open basins or closed docks are more suitable for

* This was a torpedo boat destroyer, it is true—the ill-fated "Cobra"—but the vessel and the disaster are typical of modern tendencies and their results.

adoption in a locality depends upon the range of tide and the meteorological conditions.

In an inland sea, such as the Mediterranean, which is practically tideless, an open basin will serve all the requirements of commerce, in so far as the provision of quayage, for the reception of cargo, is concerned. Nor is there much inducement to construct closed docks when the range of tide is moderate, say not exceeding about 10 feet, instances of which occur, amongst other places, at Glasgow, Belfast, and Hamburg; but when the rise and fall of the water level is very great, as at Liverpool, Bristol, and elsewhere, where there is a difference in level of between 30 and 50 feet, the necessity for enclosed areas, in which the water may be impounded at a fairly constant depth, becomes evident and imperative.

The advantages attaching to tidal basins, where practicable, are the speedy and unrestricted arrival and departure of vessels, and the absence of costly appliances for closing the entrances. On the other hand, the maintenance of an unchanging and uniform water level in tidal situations, is of undoubted benefit in facilitating the loading and discharging of cargoes, in avoiding the chafing of vessels against the quayside, and in obviating the necessity of constant attention to and alterations in the moorings.

Apart from the tidal question, an enclosed and sheltered dock has the advantage of providing a quiescent area unaffected by external waves and storms.

In a determination of the particular design suitable for a dock or basin, such great influence is exerted by considerations of a purely local nature, and there is so much scope for the exercise of individual judgment and opinion, that it is quite impossible to lay down any hard and fast rules to be observed universally, or even in a majority of cases.

Very rarely does the Engineer find himself absolutely unfettered by restrictions arising from fixed conditions, such as those relating to site, expediency, and economy. Commerce is erratic to this extent that it does not necessarily favour ports possessing admirable natural facilities for the accommodation of shipping. A port is only one of several stages in the journey from the manufacturer to the consumer. Consequently, any particular merits it may possess as a harbour, are entirely subservient to its position in regard to the great trade routes. In the maintenance of well-established lines of communication much inconvenience has been endured from natural obstacles, and large sums have been expended upon their mitigation and removal; whereas other ports, more favourably endowed by nature, have languished in obscurity. Trade, therefore, cannot be created at will; but much may be done to induce and foster it, just as it may be injured by indifference and neglect.

It is mainly, then, within areas already occupied and probably densely populated, that provision has to be made for the formation and extension of dock accommodation. In such cases the acquisition of adjoining property has to be kept within remunerative or, at any rate, strictly utilitarian limits,

and very often the new boundaries are so irregular as to need the exercise of much thought and skill in order to utilise the enclosed space to its fullest extent. Many docks owe the complexity and apparent eccentricities of their outlines to such conditions of evolution.

As, however, in a treatise of this kind we must have some basis upon which to found our observations, which are to be as complete and comprehensive as possible, there is no alternative but to assume a freedom of choice and design which will rarely, if ever, be realisable in practice. Upon such an assumption the following points claim foremost attention:—

> The most *convenient position*, and
> The most *suitable shape* for a dock;
> The best *ratio between quay space and water area;* and
> That between *periphery and surface.*

Position.—In regard to this point certain obvious requirements immediately present themselves — accessibility, shelter, accommodation. Accessibility will depend, in the first place, upon the depth of water in the approach channel. This, of course, is susceptible of improvement by artificial means, but a naturally deep fairway is a great saving in cost, both of construction and of maintenance. In the second place, accessibility will depend upon the absence of dangerous shore eddies and currents; in the third place, upon proximity to the open sea, and, lastly, upon the range and duration of the tide. The amount of shelter will be governed by the configuration of the coast line, by the vicinity or otherwise of promontories and headlands, and by local experience in the matter of storms and cyclones. The accommodation will depend upon the area available and its disposition.

Apart from considerations of exposure, a position upon the seaboard is preferable to one some distance up a river, for large ocean-going steamers. The navigation of a river, often tortuous in course and crowded with craft of various sizes, is a slow and, in fogs and darkness, a hazardous proceeding, rarely attended by any compensating advantages. Such ports as Antwerp and Bremen are undoubtedly handicapped by their inland situations. The disadvantage has perhaps not been fully apparent in the past, but it is bound to make its influence felt in the future. Joined to the difficulty of manœuvring mammoth vessels will be the attendant loss of time, which busy mercantile communities can ill afford to endure. No doubt engineering operations are quite capable of maintaining and improving the accessibility of these ports, but only at considerable outlay in initial and current expenditure. Ports like Marseilles and Havre, on the other hand, will always naturally enjoy the privilege of direct and unimpeded communication with the ocean. But it must not be overlooked that such ports are subject to the whole violence of the open sea in time of storm, and that the provision of shelter from such destructive agencies will often necessitate very expensive protective works.

Taking all things into consideration, an estuarine situation is perhaps

most to be recommended, combining, as it does, the advantages of both the previous cases without any of their drawbacks in an acute form. But, in order to fulfil the ideal conditions, the estuary must be broad and well sheltered, free from shoals and from a shallow bar.

Shape.—The outline of a dock or basin may be that of any geometrical figure, or of several figures in combination. Figures approaching the curvature of the circle, unless, indeed, the radius be extremely great, are obviously unsuitable for enclosures destined to accommodate long straight vessels in contact with their sides. Curves are undoubtedly employed to advantage in many cases, in connecting outlying arms and branches, and in training ships through changes of direction, but their effective use is limited and otherwise to be deprecated. The most suitable forms are rectilinear, and those generally available for the purpose are the triangular, the square, the rectangular, the diamond (or lozenge), the machicolated, and the digital.

The *triangular form* is rarely used, not so much, perhaps, on the ground of any inherent defect, as that the quay arrangements are not always conformable to a plan of that character. It has possible advantages for an entrance basin acting as a vestibule to a group of docks, as exemplified in the basin leading to the Albion and Island Docks at Rotherhithe (fig. 18). This example, however, be it noted, is somewhat defective, though not radically so. Other triangular outlines, more or less complete, are to be found in the Prince of Wales Dock at Swansea (fig. 20), the Morpeth Branch Dock at Birkenhead (fig. 6), and the Manchester Dock at Liverpool (fig. 5).

The *square dock* offers the advantage of plenty of space for the turning of the vessels it accommodates. In the majority of instances a vessel leaves, and should leave, a dock stem first. As she generally makes her entry in the same manner, it behoves that sufficient room be provided for turning her within the dock. This proviso is of most importance in exposed situations with narrow entrance channels. With a wide open fairway, sufficiently sheltered, it is a matter of indifference whether the turning takes place within or without the dock. Many ships will take advantage of an outer basin in order to make their entry stern first, so as to be ready for direct departure. The disadvantage attaching to the square dock is the excessive proportion of its water area to the amount of quayage, which renders it unsuitable for the accommodation of large vessels. It is doubtful whether any existing dock is absolutely square, but the Albert and Collingwood Docks, at Liverpool (fig. 5), are sufficiently close approximations for the purpose of illustration.

The *rectangular dock* is a modification of the square dock, designed to overcome the defect just mentioned. By proper manipulation the length and breadth may be arranged so as to give the maximum amount of quay frontage consistent with the water space absolutely required for manœuvring purposes. This ratio will be discussed later.

The rectangular form is common. A few instances of its adoption may

be cited from Avonmouth, Cardiff (Roath Dock), and London (West India Dock, fig. 16).

The *lozenge*, or *diamond*, is a slight deformation of the square, resulting in an improvement of form when the entrance is at one of the acute angles, as is the case in the most noteworthy instance of its use—viz., at the Empress Dock, Southampton (fig. 1).

The *machicolated form* consists of any rectilinear outline in conjunction with a number of internal projections, often of the nature of jetties or staiths. It constitutes an admirable means of utilising large docks to their fullest extent, as will be evident from an inspection of the plans of the Alexandra Dock at Hull (fig. 12), the Victoria Dock at London (fig. 17), Penarth Dock, and others.

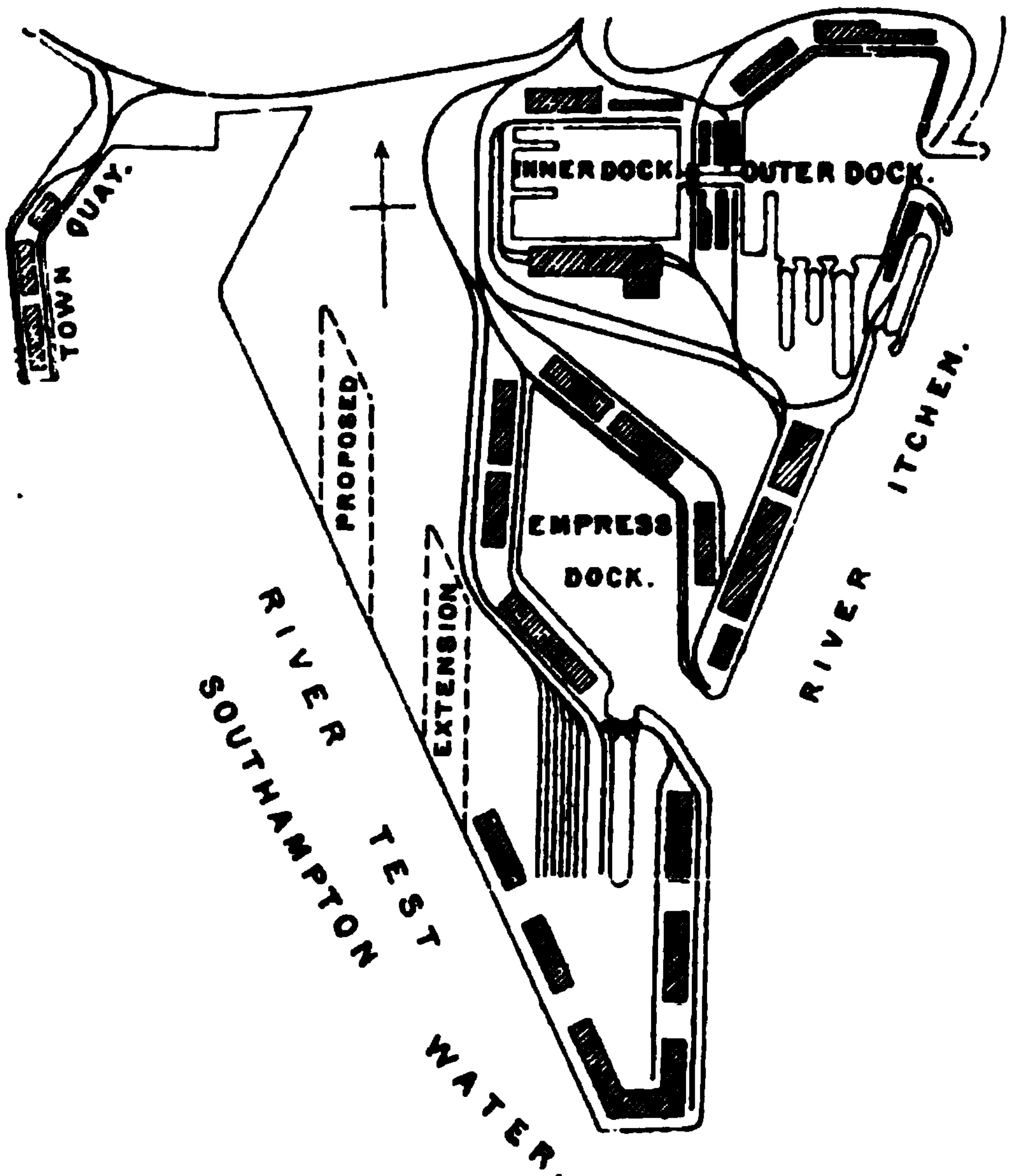

Fig. 1.—Southampton Docks. Scale, $\frac{1}{20,000}$.

A particular variation, or possibly an evolution, of the previous type is the *tridentine*, in which a main dock is provided with three important arms or branches, perpendicular to it. Such is the shape adopted for the Tilbury Docks at London (fig. 9), the Alexandra and Huskisson Docks at Liverpool (fig. 5), and the Prince's Dock at Glasgow (fig. 10). There is no essential limit to the number of branches, but three appears to be a very serviceable number consistent with compactness of design. For reasons of traffic, the branches should be arranged to the landward of the main dock.

Finally, we come to yet another evolution of the machicolated, which, from its resemblance to the outspread fingers of a hand, may appropriately

be termed the *digital*. It is illustrated in fig. 2. The suggestion emanated, in the first instance, from the late Thomas Stevenson, but the design in the figure embodies several important modifications of the original sketch, and includes an entrance scheme which has not, to the author's knowledge, appeared elsewhere. The idea is that the dock is situated on the margin of a tidal river, or estuary, and the dual entrance, as explained in Chapter vi., is intended to permit of the dock being accessible at all stages of the tide. When the flow is up the river, vessels will enter by the upstream locks and depart by the downstream locks. *Vice versâ*, when the tide is running out, incoming vessels will use the downstream locks, and those departing, the upstream locks. In this way the dock will be worked without intermission and without obstruction. It is assumed that the outer sills are deep enough to allow vessels to pass over them at low water.

The scheme has been amplified so as to include all the features essential to a dock system. Graving docks of various sizes are arranged between the entrance locks, with ample intermediate space for ship-repairing depôts. In order to have shoreward connection for these, it will be necessary for the locks to be spanned by movable bridges.

The central portion of the dock is semi-circular in form, and designed to afford turning room for vessels up to 1,000 feet in length. There are also four utilisable berths, each 275 feet long.

The branches, of which there are five, though irregular in form are all similar, and each provides quay accommodation in pairs of lengths of 1,000, 600, and 400 feet successively, together with an end berth of 350 feet. The indentations permit of ships overlapping, while at the same time berths are afforded for small craft of 100 to 120 feet in length. A further advantage of the indentations is that moored vessels are well recessed out of the way of those passing in and out of the branches; in fact, provision is made for vessels being attended in their berths by rows of lighters on each side without obstructing the main waterway.

The sides of the branches, generally, are lined with sheds, from 100 to 120 feet in width, of varying lengths, and of heights taken at two storeys, but capable of adjustment to circumstances. The sheds are recessed 40 feet from the edge of the quay, to allow of lines for quay cranes and railway trucks. These lines, as well as others at the rear of the sheds, are all in inter-communication by means of a circular railway along the landward boundary of the estate, which is supposed to be connected with trunk lines leading to other towns.

Special berths are provided at one branch dock for petroleum and coal, and at another for grain and timber. The petroleum berth has both tank storage and shed accommodation for barrels. The coal berth consists of an open quay, laid with numerous sidings and furnished with projecting jetties for hoists and tips. Grain is received direct into warehouses, the face line of which is within 5 feet of the edge of the coping. Timber may be discharged into a single storey shed or on to a low quay, or it may be floated

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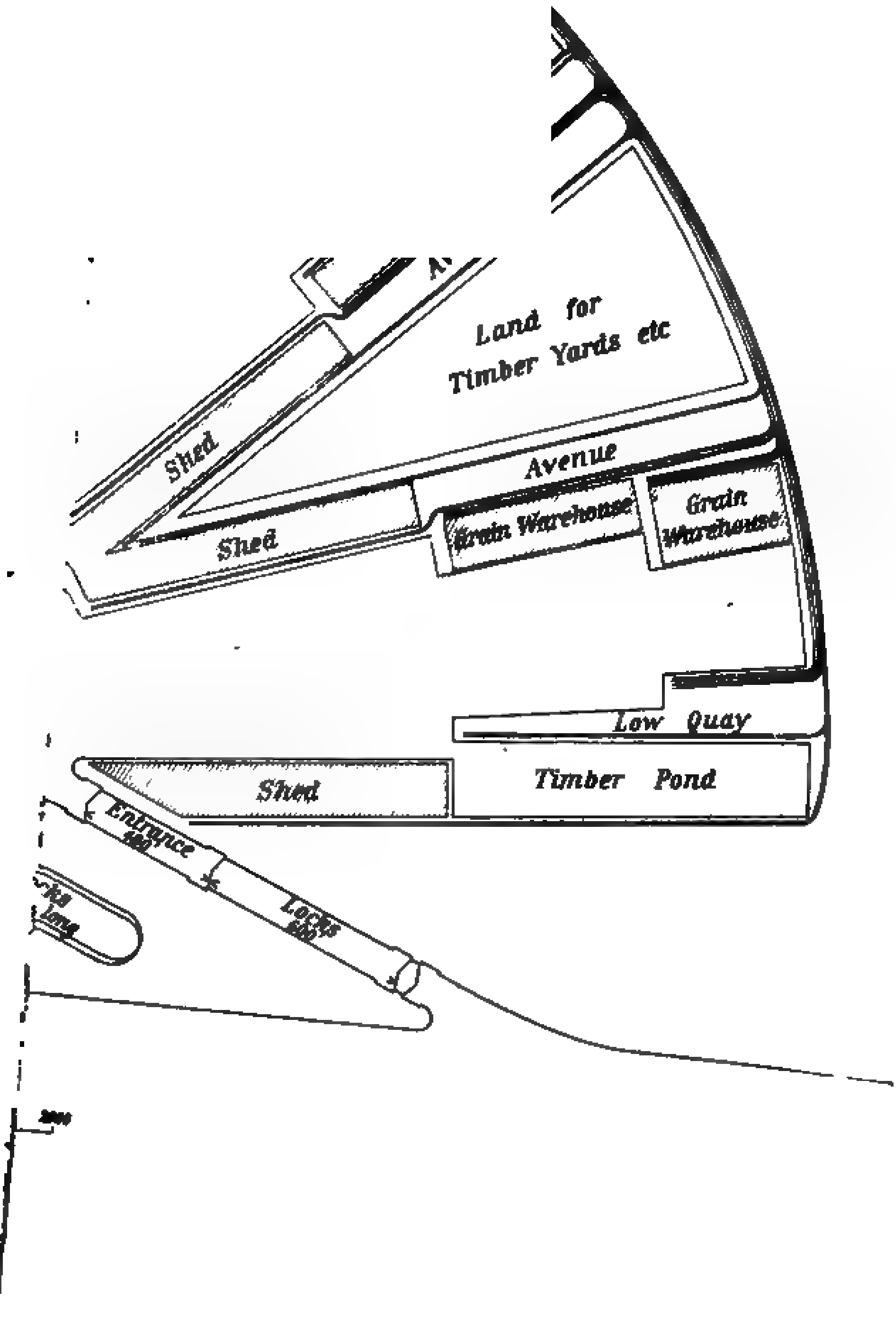

Land for
Timber Yards etc
Shed
Avenue
Shed
Grain Warehouse
Grain
Warehouse
Low Quay
Shed
Timber Pond
Entrance
Locks

into the timber pond. The river frontage is also available for timber storage, as well as for a cattle wharf, if required, with a lairage at the rear.

There are four surplus plots of land, triangular in shape, between the branches. These can be utilised as sites, partly for administrative buildings and offices, and partly for warehouses and goods depôts, timber yards, and the like commercial adjuncts of a dock system. The land immediately adjoining the entrance locks will be advantageously occupied by the dock-master's office and residence, and by dwellings for dockgatemen and other officials whose constant attendance upon the spot is desirable. A convenient site will also be found in the vicinity of the graving docks for a pumping station and, if hydraulic power is to be employed, for one or more accumulators, though possibly the requisite power may be as readily obtained from an external source, such as the mains of a private company or of a municipal body.

The design is an ideal one in this respect, that it presupposes an entire freedom of action in regard to site and outlay which is rarely attainable. There is nothing, however, to prevent the carrying out of the scheme partially or in instalments, as may be found necessary.

Ratio of Quay Space to Water Area.—The ratio of quay space to water area will depend upon the relationship between the carrying capacity and the length of vessels which occupy berths in the dock in question. The following is an approximate statement of the nett registered tonnage of vessels per lineal foot, averaged from a considerable number of cases. It must be emphasised, however, that there is much variation dependent on the design of the vessel, whether for cargo solely or for cargo and passengers combined:—

Vessels	between	200	and	300	feet long,	5 to 6	tons	per	lineal	foot.
,,	,,	300	,,	400	,,	6 to 7	,,	,,	,,	
,,	,,	400	,,	500	,,	8 to 10	,,	,,	,,	
,,	,,	500	,,	600	,,	10 to 12	,,	,,	,,	
,,	,,	600	,,	700	,,	12 to 15	,,	,,	,,	

Assuming a cubic equivalent of 40 feet to the ton, it is evident that the volume of space required for the reception of cargo will range between 200 cubic feet per lineal foot for small vessels and 600 cubic feet per lineal foot for large ships. This accommodation may be provided, either in open quay space or within covered sheds, in which latter case the available area will be doubled or trebled, if the shed have two or three storeys. But as goods will rarely be piled or stacked to a greater height than 10 feet, and as allowance must be made to the extent of 33 per cent. for alley ways and passages, it will probably be equitable to take an average of 5 feet in height over the whole surface. Accordingly, a superficies of from 40 to 120 feet per foot lineal will be required for the accommodation of cargo, but this is on the assumption that the whole is deposited upon the quay before the removal of any portion. On the other hand, no provision has been made for the simultaneous reception of outward-bound merchandise. The whole problem, in

fact, is so beset with possibilities and contingencies as to admit of no definite solution. Experience alone will demonstrate the adequacy or otherwise of the quay space appropriated to any particular vessel or class of goods.

Ratio of Periphery to Surface.—The proper proportion between the surface of a dock and its periphery is largely dependent upon the combination of length and breadth which is most suitable for the twofold purpose in view—viz., the provision of sufficient space for manœuvring ships and of sufficient quayage for berthing them. Of the two dimensions which produce the area, the length will either be the greatest available, or that, at least, which is judged adequate for present and future requirements. In assigning a breadth to a dock, it must be borne in mind that a steamship will not infrequently coal from hulks alongside during the same period in which she is receiving and discharging cargo. She may also have several lighters in attendance for goods destined to be forwarded by river or canal. In fact, employing a concrete example, it will be well to make allowance for a row of barges, 20 feet in width, to lie between the vessel and the quay and for two rows of similar craft on the other side of the vessel. Taking the beam of the latter at between 60 and 70 feet, it is evident that the berth must extend to some 120 or 130 feet in width. Doubling this for two sides, and allowing a central margin of 100 feet for the passage of ships in and out of the berths, it is clear that 350 feet is no excessive width for a dock. Indeed, an examination of Table v. will show that this dimension is frequently exceeded. At the same time, it must be observed that in cases of extreme width the dock will generally be found intersected by projecting arms or jetties.

It has already been remarked that the square form is not economical from the point of view of obtaining the greatest amount of quayage from a given area. This discrepancy is most marked in docks of large size. If the side of the dock be s, the ratio of surface to periphery is s^2 to $4s$, or s to 4, so that the disparity increases with the length of the side. In a rectangle of length, l, and breadth, b, the ratio is lb to $2l + 2b$, or if $b = nl$ where n is any proper fraction: nl^2 to $2l\ (1 + n)$, *i.e.*—

$$l : \frac{2\ (1 + n)}{n}.$$

By giving n the values, in succession, of $\frac{1}{2}$, $\frac{1}{3}$, $\frac{1}{4}$, $\frac{1}{5}$, &c., we get the following ratios:—

$$l : 6,\ 8,\ 10,\ 12.$$

When $n = 1$, the figure is a square and the ratio becomes $l : 4$ as before.

Grouped Docks.—The growth of trade being gradual, docks increase in number as circumstances at each port demand. Where a series of docks are thus brought into existence they will generally be placed in intercommunication by means of passages. Grouping can be effected systematically in various ways, as will be evident from a consideration of what may be called the "chain" system at Buenos Ayres (fig. 8), the "comb" system at

Liverpool (fig. 5), and the "barb" system at Hamburg (fig. 13). In the majority of instances, however, there is no system at all, the docks being grouped in an irregular and involved manner only explicable on the ground of unforeseen expansion.

Internal Dispositions.—The internal dispositions of a dock system have already been indicated in the description of the model plan (p. 22), but it will be advisable to enlarge a little further upon them.

In large ports it is a commendable (and even a necessary) arrangement to have separate docks for the reception of special classes of merchandise (coal, for instance, and petroleum) which it is not desirable to mix with cargo of a more general character.

A very frequent disposition at *coaling ports* is to provide along one or more sides of a dock a series of projecting coal tips, or shoots, served by lines and sidings. When one side of a dock is sufficient for the purpose, the others may be devoted to miscellaneous cargo, but the dust arising from the shipment of coal renders it advisable to conduct tipping operations as far as possible from any goods likely to be contaminated thereby. At ordinary ports where coal is shipped for fuel mainly, if not altogether, loading can be performed from hulks ranged alongside each vessel, while her cargo is being dealt with on the quay—a method which saves much time.

Petroleum is brought either in barrels or in bulk. For the latter system, which is the most general, tank steamers are essential, the oil being pumped from the steamer direct through mains to storage tanks upon the quay. On account of the extreme danger of fire, petroleum berths must be thoroughly isolated.

Grain is discharged either by small portable elevators over a ship's side into lighters and barges, or by means of stationary elevators direct into warehouses, which for this purpose are built close to the edge of the quay.

Timber used to be conveyed almost exclusively in sailing ships, and the logs were drawn out through apertures in their bows on to a low quay or into the dock. This method still prevails, but a considerable quantity of timber nowadays, particularly deals, comes by steamship, and has to be discharged from the deck or the hold in the ordinary way. On account of the great amount of quay space monopolised by timber cargoes, it is in many cases found a convenient arrangement to load the timber on to bogies or small trucks ashore, or on to large pontoons, afloat, for removal to a convenient storage ground; or, again, logs and sleepers may be formed into rafts to be floated into timber ponds.

Flour is one of the most delicate kinds of merchandise. It is very susceptible to deterioration and readily acquires a flavour from its environment. Accordingly it should not be discharged in the immediate neighbourhood of substances with strong odours, such as fresh fruit.

Cattle necessitate special wharves with isolation zones and lairages. The

regulations of the Board of Agriculture require the animals to be inspected before any part of the cargo is discharged, and to be slaughtered at the point of disembarkation.

Cost of Dock Construction.—A point of very marked, and even vital, interest to the engineer is the approximate cost of a projected undertaking, and any guidance in forming his estimates, or in affording a basis for comparison with works of a similar nature elsewhere, is readily welcomed; but information sufficiently reliable for the purpose is rarely available in dock engineering, on account of the extreme diversity of circumstances under which its operations are carried out. The cost of dock construction varies exceedingly, depending, as it does, upon such mutable conditions as the difficulties appertaining to each particular site, the current price and transport rate of material, the cost of labour, combined with an extremely wide range of equipment. Some docks have gates; others do not need them. Some are bordered by open quays; others are provided with sheds, several storeys in height. There is, in fact, absolutely no uniformity of treatment, and anything in the nature of comparison is practically impossible. The following statistics are inserted by way of interest merely. They are of no value whatever as a standard of cost in localities, and under circumstances other than those which they actually represent:—

ACTUAL COST OF DOCKS AND THEIR EQUIPMENT PER ACRE OF WATER SURFACE.

Victoria Dock, Dundee, . .	£10,600	Alexandra Dock, Liverpool, .	£23,300
Barry Docks, South Wales, .	12,950	Albert Dock, Hull, . . .	24,300
West India Docks, London, .	15,000	Queen's Dock, Glasgow, . .	24,450
Prince of Wales Dock, Swansea,	18,600	Alexandra Dock, Hull, . .	28,900
Victoria Harbour, Greenock, .	21,730	Canada Branch Dock, Liverpool,	40,000

Fresh Water Supply.—An important point in dock design, which must not be overlooked, is the provision of a supply of clean water to replenish the waste due to leakage and other causes, and also to prevent the dock from becoming foul and insanitary. The writer's experience of leakage through gates and of losses through lockage under normal circumstances at the port of Liverpool, is that the combined depression does not exceed an inch per hour over the whole water surface, but in other localities it may be more or less according to the conditions which obtain. On the sea coast and in estuaries, the tide may be relied upon to effect the necessary augmentation and changes in an efficient manner, but in rivers highly charged with sediment, such extraneous means of supply cannot be adopted without incurring considerable expense in the removal of sand and silt from the interior of the dock. In this case it is preferable to seek fresh water from some inland source to feed the dock, the water in which must always be maintained at a higher level than that of the river. Where this plan is inapplicable the difficulty may be overcome by constructing between the river and the dock a

long canal, the leisurely flow through which for a considerable distance causes the sediment to be deposited before entering the dock. The material has still to be dredged by this method, but the operation is confined to a limited space, and can be carried on without interfering with shipping. The system has been successfully tried at Calcutta (fig. 11), where the feed-canal is 3,300 yards long, and it is found that the whole of the water-borne mud brought in from the River Hooghly is deposited within the first thousand yards.

Ship Design.—The question of ship design is so much akin to that of dock design that no apology is needed for a few passing remarks upon the former subject. Within recent years very great strides have been made in naval construction, and the profile of ships has undergone a considerable change. The graceful curved outlines amidships and the deep keel of a generation ago have now given way to a square box-like section, with a flat bottom and with sides perfectly upright, or having an inward inclination towards the top. These new features, shown on fig. 3, obviously demand quays with absolutely perpendicular faces and entrances with level sills.

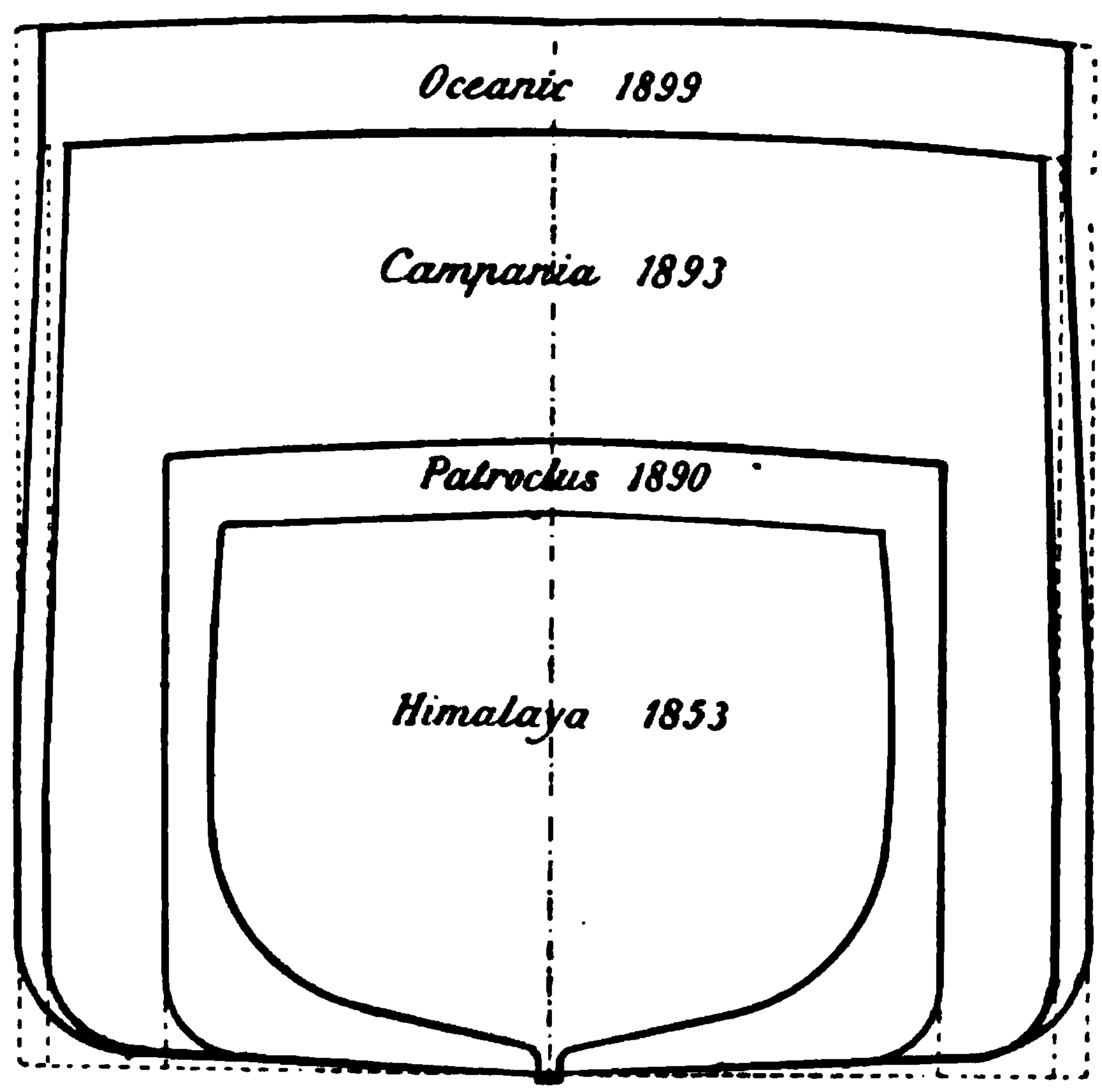


Fig. 3.—Amidship Sections of Typical Vessels.

In a paper read before the Institution of Naval Architects in 1899, Mr. G. B. Hunter * thus describes the design of a modern vessel, suitable for carrying large cargoes across the Atlantic economically and safely on a moderate draught. "With docks, harbours, and markets as they are and will be, a typical American freight steamer of the present or early future may be designed to carry not less than 12,000 tons deadweight, with cubic

* Hunter on "Large Atlantic Cargo Steamers," *Min. Proc. Inst. N.A.*, 1899.

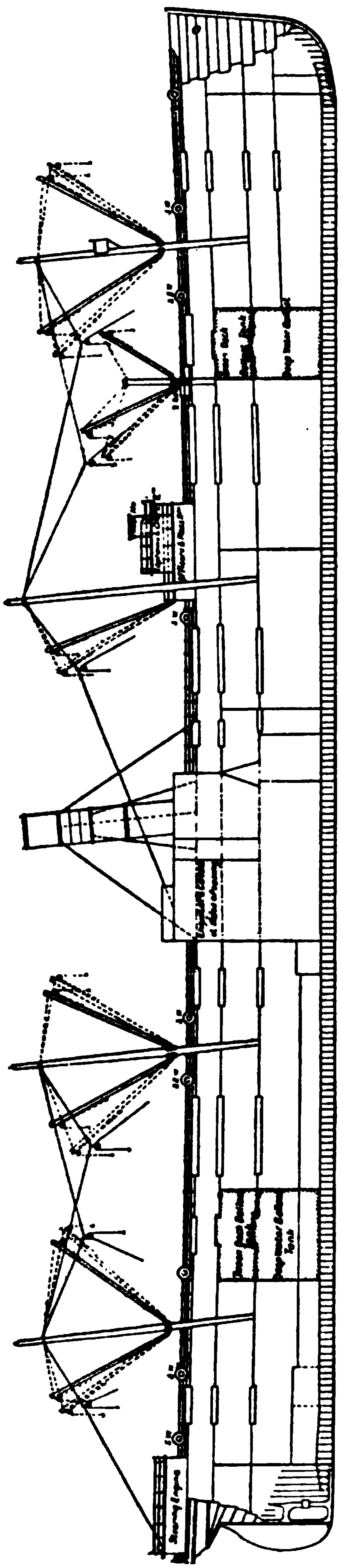


Fig. 4.—Longitudinal Section of Modern Cargo Steamer.

capacity for 20,000 tons of cargo at 40 feet per ton and 1,000 tons of fuel. This would require dimensions approximately as follows:—Length between perpendiculars, 500 feet; breadth, 60 feet; depth, moulded, 36 feet to main deck; 44 feet to shelter deck. The draught of water loaded would be about 27 feet 6 inches." The longitudinal section of such a vessel is shown in fig. 4.

These remarks were made without reference to the advent of the "Oceanic," but they will serve as the approximate standard of an average purely cargo-carrying vessel. Vessels built for passenger traffic are, of course, on somewhat different lines. Most steamships combine, in varying proportions, the functions of passenger transport with freight-carrying.

The largest vessels at present under construction are 760 feet long by 78 feet beam and 52 feet deep. There can be no doubt that even such large dimensions as these will be exceeded in the near future. The 1,000-foot vessel is almost within the range of practical politics.

Naturally, these conditions do not apply to all ports, but they serve as an indication of modern tendencies. And as it behoves a dock engineer, above all things, to exercise foresight and to be prepared for growth and expansion, he will lay his plans accordingly.

The following table gives an average of the leading dimensions of the twenty largest steamships in existence at each of the years named, between 1881 and 1901, and an approximate forecast for the year 1911:—

	1881.	1891.	1901.	1911 (forecast).
Length, . . . feet,	460	507	599	780
Breadth, ... ,,	45	$54\frac{1}{2}$	65	82
Depth, ... ,,	30	31	39	50
Loaded draught, . ,,	24	27	32	39
Tonnage, ...	4,900	6,980	14,150	26,000

We now proceed to consider the arrangements adopted under conditions actually prevailing at various ports.

Liverpool and Birkenhead Docks.

The port of Liverpool (including Bootle and Birkenhead) possesses a system of docks which for extent, completeness, and efficiency may be described as unrivalled. To what degree these results are due to its administration by one authority it is difficult to say, but there can be no doubt that the single jurisdiction of the Mersey Docks and Harbour Board, as a public trust, has conferred greater benefit upon the town and port of Liverpool than the conflicting interests of a number of separate dividend-earning companies have been able to afford to the Metropolis.

Liverpool, it must be admitted, possesses great natural advantages. The town is favourably situated close to the seaboard of the St. George's Channel, upon a wide and sheltered estuary, affording it a water frontage of over 6 miles.* It stands at the portal of the great manufacturing districts of Lancashire and the Midlands, and it is in close proximity to the coal-fields and the mineral wealth of the North of England and North Wales. Furthermore, it is linked by railways and canals with the whole of the interior of Great Britain. It is, in fact, the great door of the West, and as a port for goods, and, in a lesser degree, for passenger traffic, is the principal channel of communication with the United States and with Canada.

The tidal area of the estuary of the Mersey is about 22,500 acres, the greater portion of which is filled with a deposit of sand, resulting in about four-fifths of the area being above low water level of spring tides. The deposit is only prevented from permanent accretion and consolidation by the erratic action of the upland water, which ploughs its way to the sea in constantly changing channels. This roving disposition of the stream is looked upon in many quarters as the salvation of the port, for were the estuary to become restricted by the accumulation of sand within it, its capacity to receive tidal water would be correspondingly diminished, and the result, as regards the maintenance of the outer channel and approaches of the port, would be serious. Hence it is that the River Mersey, though by no means a model river, is left severely alone.

* The recent inclusion of Garston within the municipal area increases the amount of river frontage to 10 miles.

The deep water channel extends from New Brighton, on the left bank, at which place the river is 5,600 feet wide, to Dingle Point, on the right bank, where the width is 7,200 feet. At an intermediate point opposite the centre of Liverpool, the width becomes reduced to 3,000 feet. Throughout this distance there is ample depth of water for vessels at all stages of the tide, the depth at low water of ordinary spring tides being 70 feet, 40 feet, and 50 feet at the above-mentioned stations respectively. On the Liverpool side, unfortunately, this deep channel is bordered by a sand bank, known as the Pluckington Bank, which shoals the river bed to such an extent as to seriously diminish the value and utility of the central docks, and interfere with the use of the passengers' floating landing-stage, which flanks the river quays at this part. Various remedial expedients have been tried from time to time, but whatever success has been obtained has never been otherwise than of a temporary nature.

The navigable depth over the crest of the bar of the river at the present date is 27 feet at lowest low water of ordinary spring tides. This result has only been obtained by a vigorous policy of continuous dredging with sand pumps. Rather more than a decade ago the navigable depth was only 10 feet at lowest low water of spring tides.

The range of tide at Liverpool is 31½ feet at equinoctial springs, 27½ feet at ordinary springs, and 13 feet at ordinary neaps. The local datum is the Old Dock sill, 4 feet 8 inches below ordnance datum. The Old Dock has long since disappeared, but the level of its sill has been scrupulously preserved.

The following table presents a succint but complete statement of the extent of accommodation afforded by the Liverpool and Birkenhead Docks at the present time :—

TABLE III.

The Datum is the Level of the Old Dock Sill, which is marked on a Tide Gauge on the River Face of the Centre Pier of the Entrances to the Canning Half-Tide Dock.

Liverpool Docks.

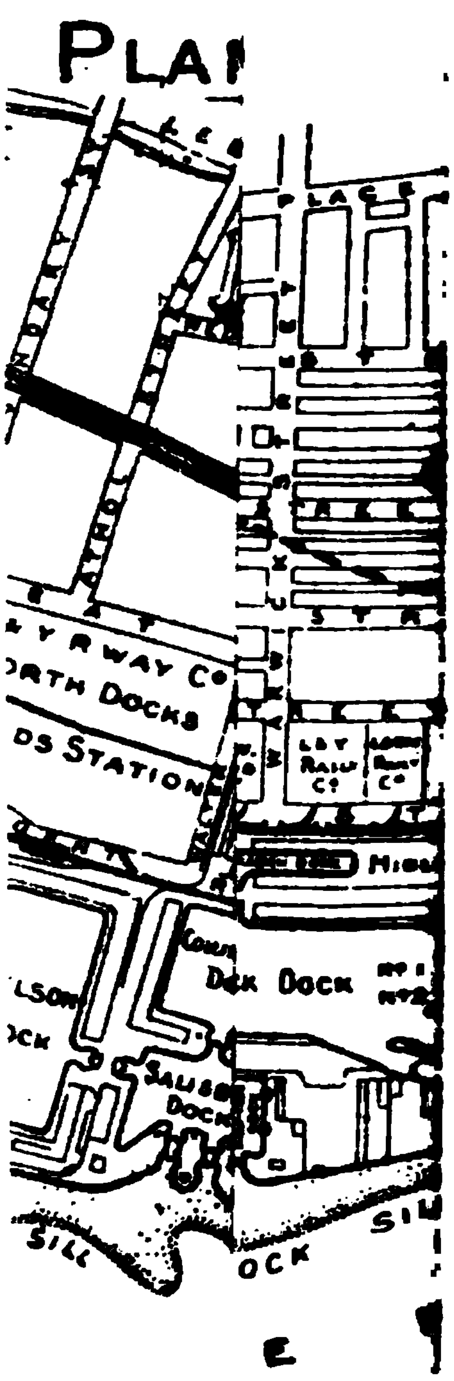
Liverpool Docks.	Position and Width of Entrance or Passage.			Sill below Datum.		Coping at Hollow Quoins above Datum.		Water Area.		Lineal Quayage.	
		Ft.	In.	Ft.	In.	Ft.	In.	Acres.	Yards.	Miles.	Yards.
Hornby Dock,	South	90	0	12	0	27	0	16	4454	0	1461
,, Branch Dock,	,,	50	0	O.D.S.		27	0	0	3354	0	308
Alexandra Dock,	S'th East	90	0	20	6	27	0	17	4281	0	1068
	S'th West	60	0	12	0	27	0				
,, Branch Dock, No. 3,	...	...		...		...		7	3420	0	846
,, ,, ,, 2,	...	...		...		...		9	2657	0	1024
,, ,, ,, 1,	...	...		...		...		9	573	0	983
Langton Dock,	S. East	50	0	9	0	27	0	18	589	0	1322
,, Lock, 238 ft. long,	S'th West	65	0	12	0	30	0	0	1719	0	160
,, ,, 119 ,,	S'th East	65	0	12	0	30	0	0	860	0	81
,, Branch Dock,	West	60	0	12	0	27	0	2	4549	0	671
Brocklebank Dock,	South	80	0	7	9	28	0	11	1010	0	1002
,, Lock, 110 ft. long,	West North	32	0	6	0	28	0	...		...	
,, ,, ,,	West Mid.	20	0	6	0	28	0	...		...	
,,	West South	60	0	7	9	28	0	...		...	
North Carriers' Dock,	West	40	0	6	0	27	0	2	3423	0	641
South ,,	,,	40	0	6	0	27	0	1	4515	0	615
Canada Lock, 600 ft. long,	North	100	0	14	0	28	0	1	2018	0	469
(a) ,, Dock,	South	90	0	14	0	28	0	24	913	0	1595
,, Branch Dock, No. 1,	...	...		...		...		7	2313	0	823
Huskisson Dock,	South	90	0	20	6	31	0	12	4273	0	711
,, Branch Dock, No. 3,	...	...		...		...		8	780	0	990
,, ,, ,, 2,	...	...		...		...		7	592	0	910
,, ,, ,, 1,	...	...		...		...		9	1125	0	983
Sandon Half-Tide Dock,	...	...		...		...		14	466	0	1081
,, (Lock, 130 ft. long),	West North	80	0	20	6	35	0	...		...	
,, (,, 165 ,,),	West Mid.	40	0	16	0	35	0	...		...	
,, (,, 130 ,,),	West South	100	0	20	6	35	0	...		...	
,, Dock,	West	90	0	20	6	31	0	10	100	0	867
Wellington Lock,	,,	70	0	6	6	31	0	7	4120	0	820
Bramley-Moore Dock,	North	60	0	6	0	26	0	9	3106	0	935
	South	60	0	6	0	26	0				
Nelson Dock,	,,	60	0	6	6	26	0	7	4786	0	803
Canal Basin, Lightbody Street,	Passage	18	0	O.D.S.		26	0	0	920	0	110
				Above							
Stanley Lock,	West	18	0	2	6	29	0	...		...	
Collingwood Lock,	,,	18	0	2	6	26	0	...		...	
Salisbury Lock,	,,	18	0	...		26	0	...		...	
	Inner Sill	...		2	6	...		...		...	
				Below							
	Outer ,,	...		5	0	...		...		...	
Stanley Dock,	West	51	0	5	5	29	0	3	3343	0	562
Collingwood Dock,	,,	60	0	6	9	26	0	5	244	0	553
Salisbury Dock,	West North	60	0	6	11	26	0	3	2146	0	406
	West South	50	0	6	11	26	0				

Note (*a*).—The water in the Canada and Huskisson Docks can be raised to give any additional depth of water required in the Canada Dock and its South Passage, so that that passage may have the same effective depth as the river entrances to Sandon Half-Tide Dock—viz., 20 feet 6 inches below Old Dock Sill.

TABLE III. (*Continued*).—LIVERPOOL DOCKS.

Liverpool Docks.	Position and Width of Entrance or Passage.			Sill below Datum.		Coping at Hollow Quoins above Datum.		Water Area.		Lineal Quayage.	
		Ft.	In.	Ft.	In.	Ft.	In.	Acres.	Yards.	Miles.	Yards.
Clarence Graving Dock Basin,	North .	45	0	4	9	26	0	1	1056	0	291
	South .	44	6	4	6	26	6				
,, Half-Tide Dock, .	West .	50	0	5	0	26	8	4	1794	0	635
,, Dock, ...	,, .	47	0	3	2	26	0	6	273	0	914
Trafalgar Lock, . . .	North .	45	0	6	7	23	10	0	2937	0	256
,, Dock, ...	,, .	44	3	6	7	21	11	6	459	0	724
Victoria Dock, . . .	South .	50	0	6	6	26	0	5	4374	0	701
West Waterloo Dock, . .	,, .	60	0	8	0	22	1	3	2146	0	533
East ,, ,, . .	,, .	60	0	8	0	22	1	2	3375	0	506
Prince's Half-Tide Dock, .	West North	65	0	8	0	31	0	4	3250	0	434
,, Lock, 110 ft. long, .	West Mid.	32	0	8	0	31	0				
,, ...	West South	65	0	8	0	31	0				
,, Dock, . . .	North .	45	0	5	11	27	5	11	1490	0	1194
*George's Dock Passage, .	South .	40	3	4	6	24	5	0	1033	0	152
Manchester Dock, . . .	West .	32	10	Above 0	3	23	3	1	595	0	339
,, Lock, 86 ft. long,	,, .	33	8	Below 3	9	24	3	0	315	0	57
*Canning Dock, . . .	,, .	45	0	6	1	26	2	4	376	0	585
* ,, Half-Tide Dock, .	West North	45	0	6	3	28	3	2	2688	0	429
	West South	45	0	6	3	28	3				
*Albert Dock, . .	North .	45	0	6	4	26	0	7	3542	0	885
	East .	45	0	6	0	26	0				
*Salthouse Dock, . .	North .	45	0	6	0	26	0	6	2019	0	784
*Wapping Basin, . .	,, .	50	0	5	8	26	0	1	3151	0	454
	South .	50	0	6	0	26	0				
	West .	40	0	5	8	25	0				
Duke's Dock, . . .	,, .	40	0	4	2	25	10	2	1336	0	1138
	Middle	40	0	4	7	22	9				
*Wapping Dock, . . .	South .	50	0	6	0	26	0	5	499	0	815
*King's Dock, . . .	East .	50	0	6	0	26	0	7	3896	0	875
Queen's Half-Tide Dock, .	...	...		...		...		3	3542	0	445
* ,, Dock, . . .	South .	100	0	17	6	29	5	10	3124	0	1114
,, Branch Dock, No. 1, .	...	...		...		...		4	4384	0	654
*Coburg Dock, . . .	West .	70	0	5	7	30	6	7	3157	0	939
*Brunswick Dock, . . .	North .	100	0	17	6	29	0	12	3533	0	1070
,, Half-Tide Dock, .	West .	45	0	6	0	26	6	1	1399	0	250
*Union Dock, . . .	North .	60	0	6	6	27	0	1	1941	0	361
	South .	60	0	12	0	31	0				
Toxteth Dock, . . .	,, .	60	0	12	0	31	0	11	1075	0	1134
,, Lock, 177 ft. long, .	West .	50	0	8	0	31	0	0	1013	0	118
Harrington Dock, . . .	South .	60	0	12	0	31	0	9	256	0	1023
,, Lock, 131 ft. long,	West .	22	0	5	9	31	0	0	320	...	
Herculaneum Dock, . .	West North	80	0	12	0	31	0	7	2581	0	596
	West South	60	0	12	0	31	0				
,, Branch Dock, .	...	...		...		...		2	853	0	577
Total Water Area and Lineal Quayage of the Liverpool Docks,								389	3751	24	1542

* The water in the group of Docks from Canning to Brunswick Docks, inclusive, is impounded over low neap tides, and any loss made good by pumping from the river. By these means the effective depth of these docks is made not less than that of the lowest Sills over which they can be approached—viz., 12 feet below datum.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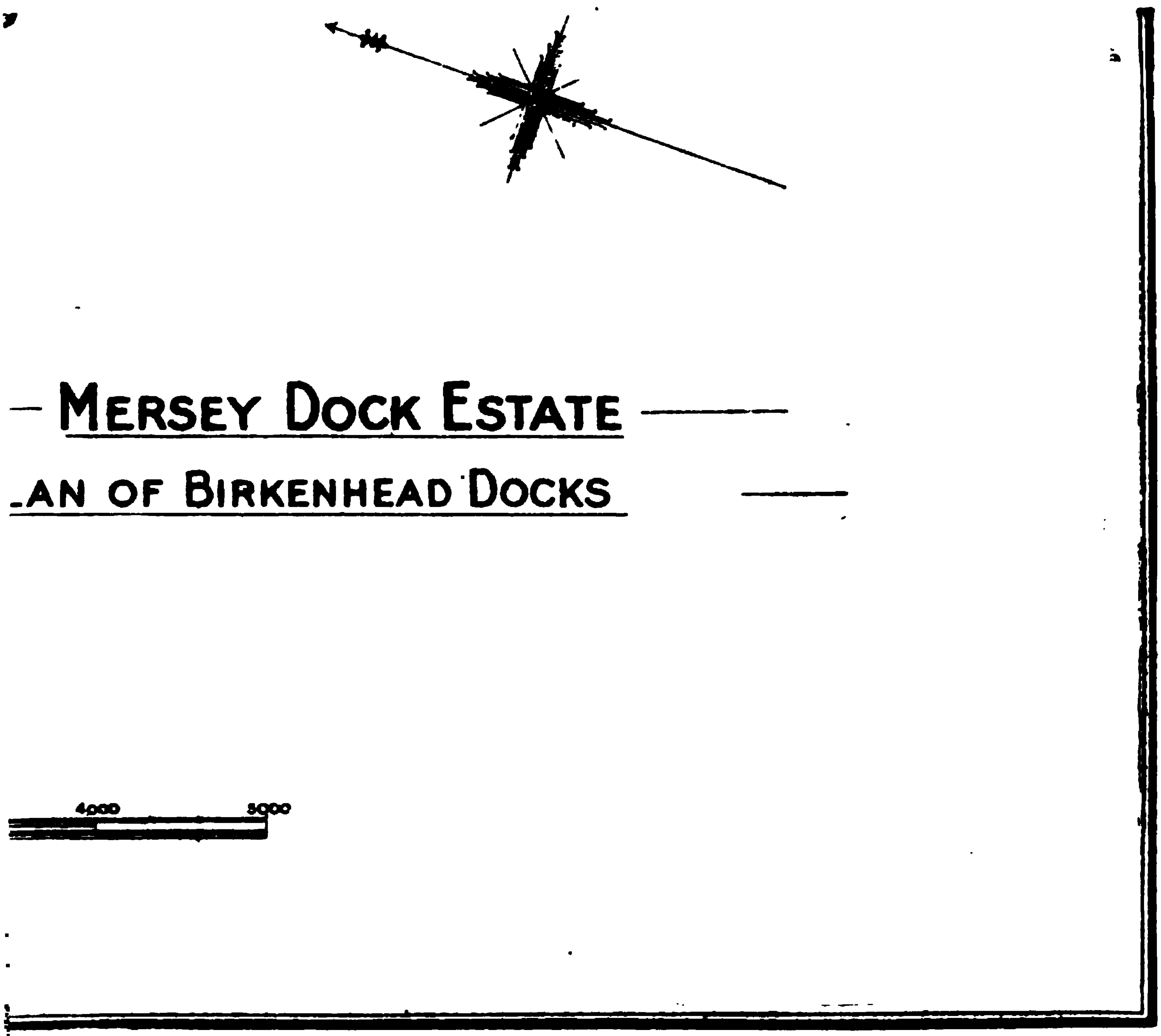

MERSEY DOCK ESTATE
-AN OF BIRKENHEAD DOCKS
4000
5000

TABLE III. (*Continued*).—LIVERPOOL BASINS.

Liverpool Basins.	Width of Entrance.		Height of Piers above Datum.			Water Area.		Lineal Quayage.	
	Ft.	In.		Ft.	In.	Acres.	Yards.	Miles.	Yards.
Canada Basin, ...	390	0	N	30	0	9	2805	0	846
			S	32	0				
George's Ferry Basin, ...	67	0		23	8	0	1344	0	160
Chester Basin, ...	36	0		22	2	0	2568	0	288
Anderton Basin, ...	46	0		25	7	0	1422	0	175
South Ferry Basin, ...	60	0		30	6	0	2927	0	205
Total Water Area and Lineal Quayage of the Liverpool Basins,						11	1386	0	1674
,, ,, ,, ,, Docks,						389	3751	24	1542
Total, ...						401	297	25	1456

BIRKENHEAD DOCKS.

Birkenhead Docks.	Position and Width of Entrance or Passage.			Sill below Datum.		Coping at Hollow Quoins above Datum.		Water Area.		Lineal Quayage.	
		Ft.	In.	Ft.	In.	Ft.	In.	Acres.	Yards.	Miles.	Yards.
West Float, ...	East .	100	0	7	6	26	6	52	319	2	210
Basins near Canada Works—											
West Basin, ...	North .	50	0	...		...		1	2554	0	543
East ,, ...	,, .	50	0	...		...		1	84	0	390
East Float, ...	...	...		...		...		59	3786	1	1673
Corn Warehouse Dock, . .	South .	30	0	O.D.S.		26	0	1	453	0	555
Railway Companies' Basin, .	...	...		...		...		0	606	0	113
Wallasey Dock, ...	...	...		...		...		12	3813	0	1261
				Below							
Passage to Wallasey Dock, .	West .	49	2	9	0	26	0	0	1333	0	234
Inner Northern Entrances, .	North .	100	0	9	0	26	0	...		0	242
Lock, 198 ft. long, . .	Middle	30	0	...		26	0	0	667	0	264
Inner Sill, ...	...	...		9	0	...		...		...	
Outer ,, ...	...	...		12	0	...		...		...	
Lock, 274 ft. long, . .	South .	50	0	...		26	0	0	1522	0	300
Inner Sill, ...	...	...		9	0	...		...		...	
Outer ,, ...	...	...		12	0	...		...		...	
Alfred Dock, ...	...	...		...		...		8	3276	0	511
Outer Northern Entrances—											
Lock, 480 ft. long, . .	North .	100	0	18	6	31	0	0	3888	0	352
,, 198 ,, . .	Middle	30	0	12	0	26	0	0	667	0	377
,, 398 ,, . .	South .	49	8	12	0	26	0	0	2222	0	391
Egerton Dock, ...	West .	70	0	7	4	25	0	4	469	0	704
Morpeth Dock, ...	,, .	70	0	5	5	25	0	11	2404	0	1299
,, Lock, 398 ft. long, .	East .	85	0	12	0	26	0	0	3777	0	441
Railway Company's Basin, .	South .	25	0	O.D.S.		26	0	0	3144	0	319
Morpeth Branch Dock, . .	West .	85	0	...		26	0	4	243	0	637
Total Water Area and Lineal Quayage of the Birkenhead Docks,								160	1347	9	256

NOTE.—The water in the Birkenhead Docks is impounded over low neap tides, and any loss made good by pumping from the river. By these means the effective depth of these docks is made not less than that of the lowest Sills over which they can be approached—viz., 18½ feet below datum.

TABLE III. (*Continued*).—Birkenhead Basin.

Birkenhead Basin.	Width of Entrance.	Height of Piers above Datum.	Water Area.	Lineal Quayage.
	Ft. In.	Ft. In.	Acres. Yards.	Miles. Yards.
North Basin, ...	500 0	31 0	4 2843	0 669
Total Water Area and Lineal Quayage of the Birkenhead Basin,			4 2843	0 669
,, ,, ,, ,, Docks,			160 1347	9 256
Total, ...			164 4190	9 925

Barry Docks.

The special feature of the Barry Docks is the accommodation provided, almost exclusively, for the coal and timber trades, and it is on this account, principally, that these docks have been selected for illustration. The town, which is of quite modern growth, having developed from a population of 100 in 1884 to one of 30,000 in 1902, is situated at the southernmost point of the Welsh coast-line, forming an outlet from the coalfields in that locality, in close contiguity to Cardiff, Newport, and Swansea.

The docks are the property of the Barry Railway Company. The entrance lies under the convenient shelter afforded by the high land of Barry Island, which protects it from westerly and south-westerly winds. The only points of exposure—viz., to the southward with a sea-range of 14 miles and to the south-east with a sea-range of 16 miles—are covered by breakwaters. There is also good anchorage, extending from Barry Island to Sully Island, a distance of 3 miles.

The range of tide at Barry is 36 feet at ordinary springs and 20 feet at ordinary neaps.

The shipment of coal takes place at the north side of both No. 1 and No. 2 docks, and on both sides of the mole in the former dock. It is stated that a steamer has entered the dock, loaded 1,900 tons of coal, and left again on the same tide.

Table iv. gives all the particulars necessary for following the arrangements exhibited in the plan.

The timber trade is accommodated at the east end of No. 2 dock, where there are two timber ponds of 6 and 35 acres respectively. Railways are provided alongside, so that timber can be loaded direct from the ponds into the railway waggons.

Fig. 7.—Barry Docks.

TABLE IV.—Barry Docks.

Description.	Area.	Length.	Width.	Quayage.	Width of Entrance Gates.	Depth of Water on Sill. h.w.o.s.t.	Depth of Water on Sill. h.w.o.n.t.	Depth of Water on Sills, Outer and Middle Gates.			l.w.o.n.t.	Depth of Water on Sill, Inner Gate. h.w.o.s.t.	Depth of Water on Sill, Inner Gate. h.w.o.n.t.
	Acres.	Feet.	Feet.	Feet.	Feet.	Ft. Ins.	Ft. Ins.	Ft. Ins.	Ft. Ins.	Ft. Ins.	Ft. Ins.	Ft. Ins.	Ft. Ins.
Dock No. 1, . . .	73	3,100	1,100	10,500	80	37 8	29 4						
" No. 2, . . .	34	3,320	400-600	7,000	80	37 8	29 4						
" No. 3 (Basin), .	7	600	500	2,040	80	37 8	29 4						
Lady Windsor Lock,	...	647	65	...	65	...	...	49 8	13 7	41 4	21 8	37 8	29 4
Timber Float, No. 1,	6	800	400	2,000	...	...	...						
" " No. 2,	35	1,600	1,100	5,000	...	...	...						
Entrance channel from the breakwater heads to deep									In Channel.				
lock gates, . .	...	1,500	...	...	...	...	...	53 8	17 7	45 4	25 8		

Note.—The sills being curved, the depth of water at the centre of the sills of the dock, basin, and deep lock entrance is 3 feet more than that given above.

Docks at Buenos Ayres.

This system of docks exemplifies (see fig. 8) the case in which an enclosed basin has been rendered necessary by other than strictly tidal reasons. The average range of tide does not exceed 2 feet $7\frac{1}{2}$ inches, the highest recorded for four years being 3 feet in the month of December, and the lowest 2 feet $3\frac{1}{2}$ inches in the month of June. The flood waters of the "Santa Rosa," however, cause the water to rise to a height of 8 feet above their normal height, and the river has been known to fall below zero to the same extent on one occasion at least. The principal reasons, therefore, which operated in favour of entrance locks are thus set forth by Mr. Dobson* :—

"In the first place, the gates were provided, not so much with the object of maintaining the water in the docks at a nearly constant level, as for the purpose of preventing it from falling below the level of zero or low water, thereby enabling vessels always to remain afloat in the docks, and, at the same time, to allow all ships of light draught to leave them, if necessary, when the level of the river was below zero. In the second place, the southernmost pair of gates, which point outwards, are constructed with the object of preventing the water of the Riachuelo, when in a turbid

* Dobson on "Buenos Ayres Harbour Works," *Min. Proc. Inst. C.E.*, vol. cxxxviii.

Fig. 8.—Docks at Buenos Ayres. Scale, [illegible].

state, from entering the docks, and also to enable the docks to be more thoroughly sluiced, if required. By closing these gates at low water the rising tide would be compelled to enter at the north end, and then by closing the north lock-gates at high water the falling tide would have to run out through the south lock, thus entirely changing the water in the docks and preventing the possibility of its becoming stagnant and consequently dangerous to the health of the city.

"Since the works have been completed and the north basin opened it has been found, from experiments made with floats, that the current in the docks is even better than was anticipated, and that, with a good average tide and a strong north or south wind, there is as much as 850 feet per hour, so that the closing of the gates in order to sluice will be only occasionally needed."

The following is a statement of the dimensions of the various docks :—

TABLE IV*a*.—DIMENSIONS OF THE MADERO DOCKS, BUENOS AYRES.

	Length.	Breadth.	Area.	Quayage.	Depth below Zero.
	Yards.	Yards.	Acres.	Yards.	Feet.
Dock No. 1, . .	623	175	23	1,553	$23\frac{3}{4}$
Dock No. 2, . .	623	175	23	1,553	$23\frac{3}{4}$
Dock No. 3, . .	755	175	27	1,815	$23\frac{3}{4}$
Dock No. 4, . .	689	175	25	1,679	$23\frac{3}{4}$

The south basin has an area of 35 acres and a depth of water of $21\frac{1}{4}$ feet below zero. The south lock is 443 feet long and $65\frac{1}{2}$ feet wide at coping level, with sills $21\frac{3}{4}$ feet below zero. The north lock is $508\frac{1}{2}$ feet long, 82 feet wide, with sills 22 feet below zero. The north basin has an area of 41 acres and a depth of $21\frac{1}{4}$ feet. The total water area of the two basins and four docks is 174 acres, and the total quayage 9,276 yards.

TILBURY DOCKS, LONDON.

These docks (see fig. 9) are planned on the tridentine system. They are approached by means of a tidal basin, having an area of $17\frac{1}{2}$ acres, with an entrance from the River Thames, 364 feet in width, flanked on each side by splayed timber jetties. Landing places for passengers are provided in the tidal basin, so that they may disembark before a vessel enters the dock. There is also a coaling jetty. The lock between the tidal basin and the inner docks is 946 feet long over all, with two chambers 555 feet and 145 feet long respectively, both 80 feet wide, and with a depth of 44 feet below Trinity high water mark on the outer and intermediate sills. The main and branch docks have a water area of $52\frac{1}{2}$ acres, with quayage accommodation for 31 vessels, averaging 400 feet in length. Each berth in the branch docks is provided with a shed, 301 feet long and 120 feet wide, and has direct

communication with the London, Tilbury, and Southend Railway. There are two graving docks placed parallel to the lock and of the same extreme length. They can be entered either from the dock or from the basin, and

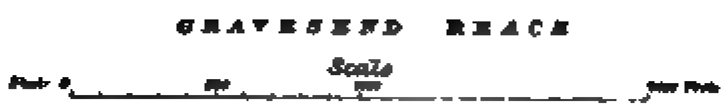


Fig. 9.—Tilbury Docks, London.

can, if necessary, be used as auxiliary locks. The depths of the basin and lock are so arranged that a ship drawing 23 feet of water can, even at low water of spring tides, proceed direct to her berth in the inner docks.

Glasgow Docks.

The dock system at Glasgow is an exemplification, also on the tridentine principle, of the method adopted in localities where gates are not rendered necessary by any considerations. The "docks" are, in fact, strictly speaking, tidal basins.

The docks are in two groups. On the right hand bank, or north side of the Clyde, the Queen's Dock has a water area of 33¾ acres, with 3,334 lineal yards of quay frontage, and a depth of 20 feet at low water. The entrance is 100 feet wide, and it is spanned by a single leaf swing bridge. The dock is subdivided into an outer or canting basin, 1,000 feet long by 695 feet wide, and two inner basins, the one 1,891 feet long by 270 feet wide, and the other 1,668 feet long by 230 feet wide, separated by a pier 195 feet wide.

On the south side of the Clyde the Prince's Dock has a total water area of 35 acres, with 3,737 lineal yards of quay frontage. The canting basin is 1,150 feet long, with a width of from 505 to 676 feet and there are three branch basins, each 200 feet wide, and 1,168 feet, 1,461 feet, and 1,528 feet long respectively. The north basin has a depth of 20 feet, the centre and south basins, 25 feet, and the outer basin, 20 to 28 feet below low water. The entrance is bell-mouthed in shape, with a minimum width of 156 feet, and is not crossed by a bridge.*

The Kidderpur Docks, Calcutta.

The tidal and fluvial conditions prevailing in the River Hooghly are irregular and conflicting. The port of Calcutta is situated some 90 miles from the sea, but the tides, when not checked by freshets during rains, exert their influence beyond that distance. "From March to July, when strong southerly winds prevail, the current at spring tides during the early portion of the floods attains a velocity of 5 to 6 miles an hour. During the rainy season, when the discharge of fresh water by the branches from the Ganges is considerable, the down-stream current during the ebb tide runs at about the same rate; and during heavy freshets in the river, the upward current at the flood tide is hardly perceptible, although the level of the water is raised for many miles above Calcutta. At neap tides there is no up-stream current at all if there are freshets; the water is headed up and the level rises, but the current is always down stream. During the rains the spring tides rise to a mean height of 20½ feet, and fall to 8½ feet above (zero) datum, while neap tides rise to 15 feet and fall to about 10 feet above datum. In the dry season, which lasts from November to June, the spring tides rise to an average height of 15 feet and fall to 2½ feet; while neap tides rise on an average to 12 feet and fall to 5 feet above datum. The tidal

* Alston on "The River Clyde and the Harbour of Glasgow," International Engineering Congress, Glasgow, 1901.

Fig. 10.

range between low water of spring tides in the dry season, and the average high water in the rainy season is about 18 feet, but during heavy floods has been as much as 22½ feet."*

Such conflicting conditions call for a special arrangement of dock entrances to permit of vessels entering or leaving on the flood tide, or when the current in the river is continuously down stream, and the arrangement adopted is shown in fig. 11. It consists of a lock, 400 feet long by 60 feet wide, and a single entrance, 80 feet wide, pointing in opposite directions, the reasons for and advantages of which are fully discussed in Chapter vi.

The half-tide basin is 600 feet by 680 feet, and No. 1 dock is 2,600 feet long by 600 feet wide, with a water area of 34½ acres.

Fig. 11.—Kidderpur Docks, Calcutta.

THE ALEXANDRA DOCK, HULL.

This dock is selected as an example of the machicolated system. It is situated near the mouth of the River Humber, has a water area of 46½ acres, a quayage of 2 miles, covering 160 acres; and is provided with a lock, 550 feet long by 85 feet wide, and two graving docks. The entrance to the lock is splayed.

"The navigable channel of the Humber approaches close to the northern shore in front of Hull; but at the Alexandra Dock the northern edge of the deep channel was 960 feet from the outer lock sill. The channel

* Bruce on "The Kidderpur Docks, Calcutta," *Min. Proc. Inst., C.E.*, vol. cxxi.

there is 35 to 40 feet deep at low water spring tides, having an almost vertical face in places on its northern side, the depth increasing suddenly from 5 to 30 feet, owing to the scour of the tidal current against this side, the channel having been eroded by it out of the hard clay of the 'Hebbles,' a shoal extending $\frac{1}{2}$ mile above and 2 miles below the Alexandra Dock. The Hebbles shoal is mostly composed of very hard boulder clay, with large

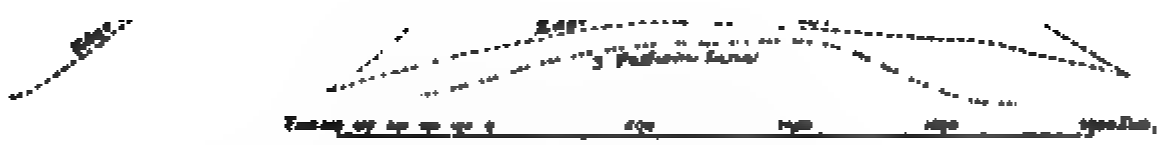


Fig. 12.—Alexandra Dock, Hull.

boulders reaching up to $\frac{1}{2}$ ton in weight, and smaller stones strewn over the surface, and beds of peat were also found."* The original surface of the foreshore and river bed and the deepening effected by the dredging operations previous to the opening of the dock for traffic are shown on the plan in fig. 12.

Hamburg Docks.

The town of Hamburg is situated 62 miles above the outlet of the river Elbe. As in the case of Glasgow, the range of tide, which averages 6 feet, is not sufficient to render gates an absolute necessity, and they have been dispensed with, although the maximum difference between high and low water reaches $19\frac{1}{2}$ feet. One reason which operated in favour of this decision was that locks would have seriously hindered the considerable traffic between sea-going ships and the river boats which ply between

* Hurtzig on "The Alexandra Dock, Hull," *Min. Proc. Inst. C.E.*, vol. xcii.

Hamburg and Bohemia. Again, the Elbe leaves little deposit near its tidal limit, so that no need for gates arises from this cause.

Fig. 13—Hamburg Docks. Scale, [illegible].

The basins B, C, and D are now practically completed. Basin B is surrounded by sloping sides with supporting pilework at the foot and with masonry jetties at intervals. Basins C and D are lined throughout with quay walls.

Various Ports.

As additional examples of the variations in dock design, a number of diagrams are here given, showing arrangements adopted at the ports of London, Sunderland, Swansea, Havre, and Marseilles.

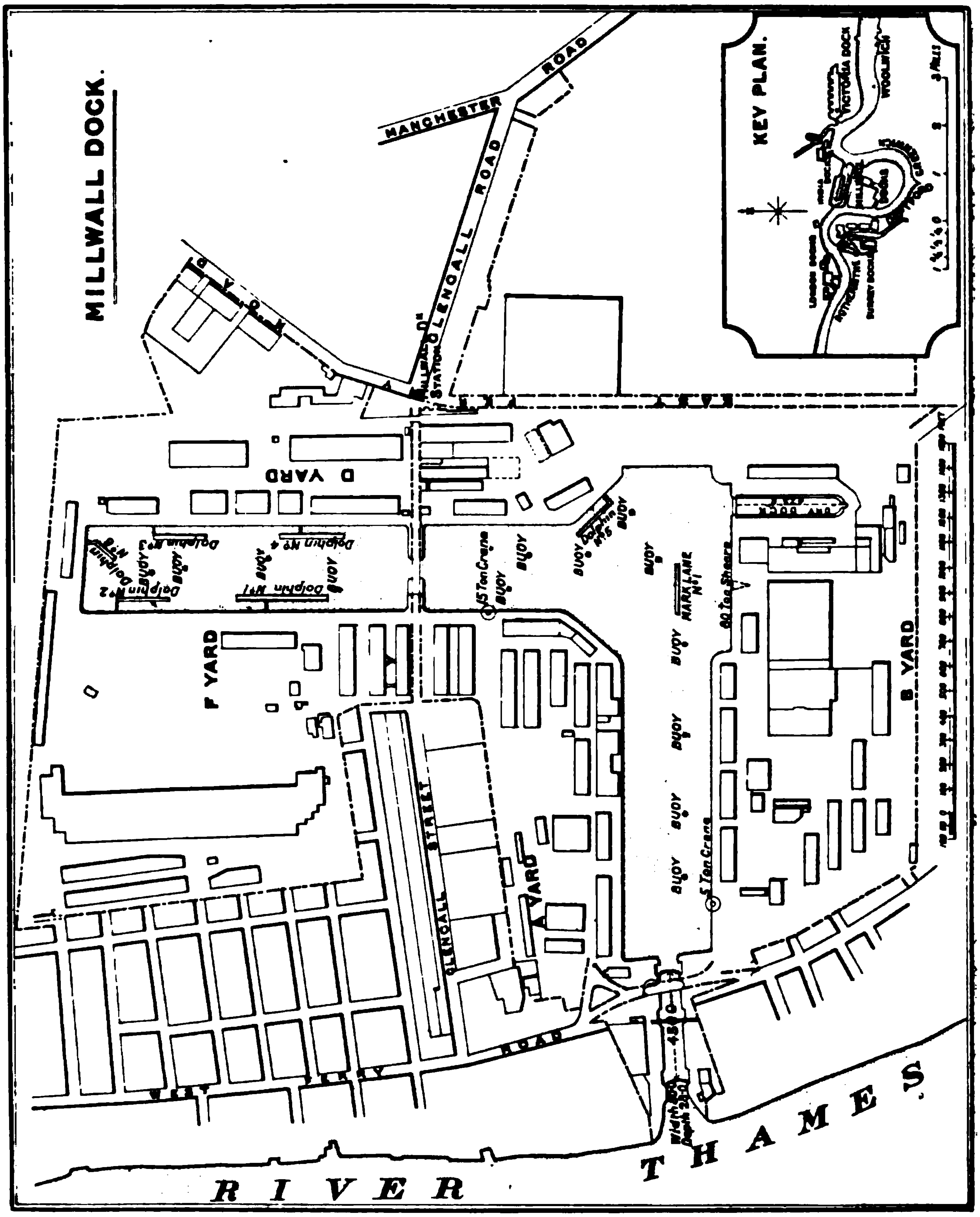


Fig. 14.

Appended is also a table giving statistics of representative docks in the British Isles and throughout the world (pp. 54, 55).

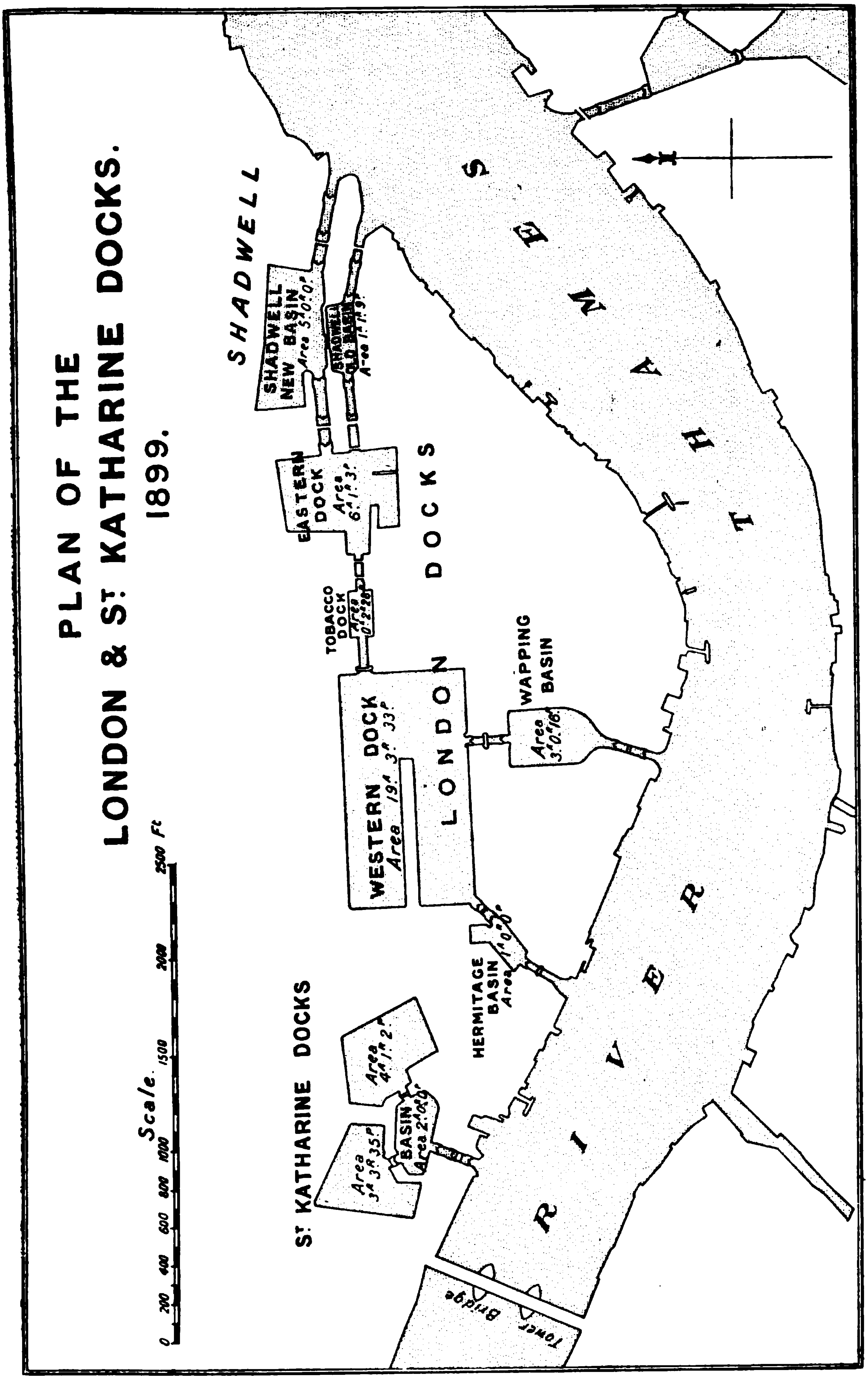

PLAN OF THE
LONDON & ST KATHARINE DOCKS.
1899.
SHADWELL
SHADWELL NEW BASIN Area 3A 0R 0P
SHADWELL OLD BASIN Area 1A 1R 19P
EASTERN DOCK Area 6A 1R 3P
TOBACCO DOCK Area 0A 2R 28P
WESTERN DOCK Area 19A 3R 33P
LONDON DOCKS
WAPPING BASIN Area 3A 0R 16P
HERMITAGE BASIN Area 1A 0R 3P
ST KATHARINE DOCKS
Area 4A 1R 2P
BASIN Area 2A 0R 6P
Area 3A 3R 35P
RIVER THAMES
Tower Bridge
Scale
0 200 400 600 800 1000 1500 2000 2500 Ft

Fig. 15.

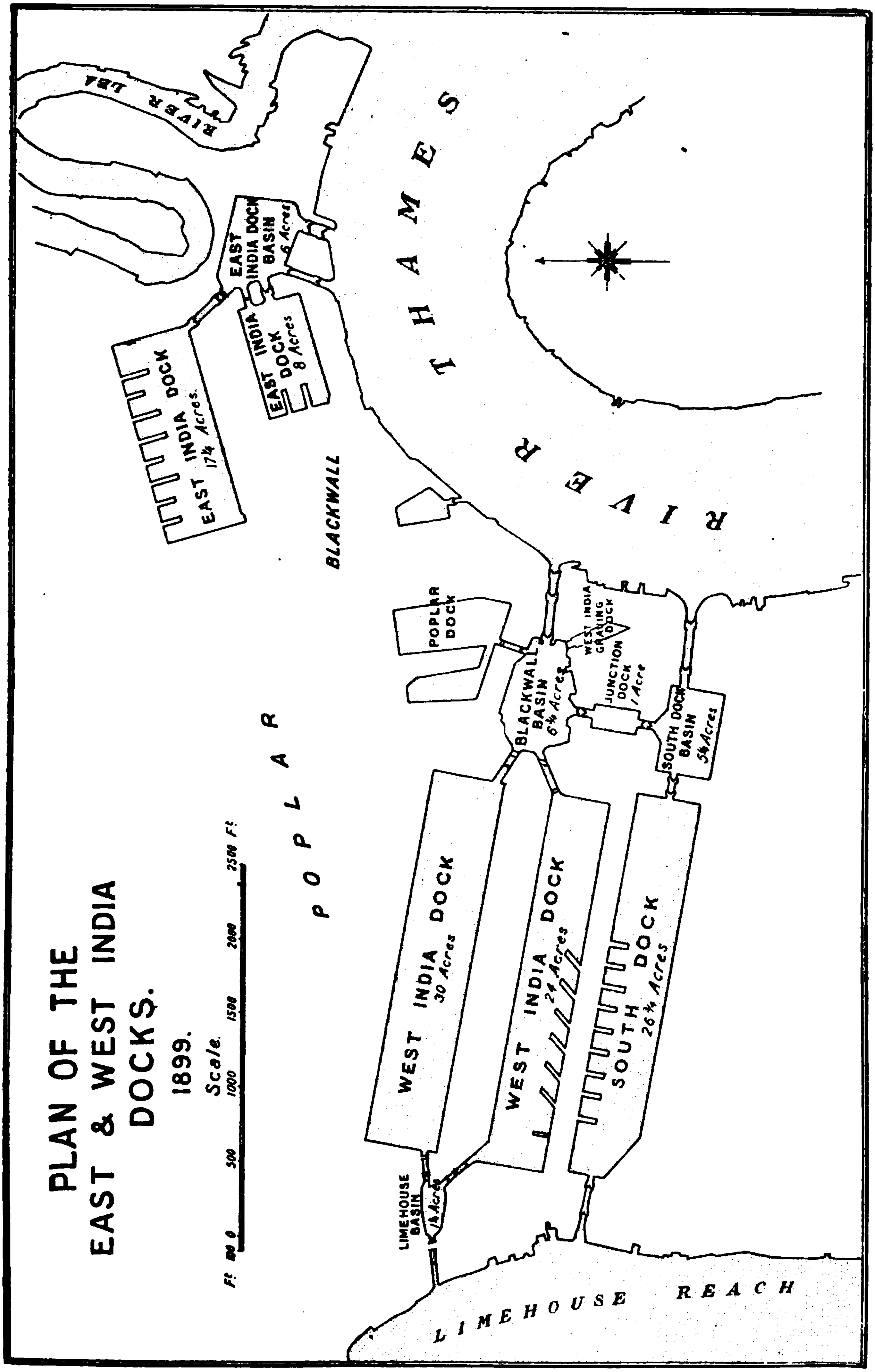

PLAN OF THE
EAST & WEST INDIA
DOCKS.
1899.
Scale.
Ft 500 0
500
1000
1500
2000
2500 Ft
RIVER LEA
EAST INDIA DOCK BASIN
6 Acres
EAST INDIA DOCK
8 Acres
EAST INDIA DOCK
17¼ Acres
BLACKWALL
RIVER THAMES
POPLAR DOCK
WEST INDIA GRAVING DOCK
JUNCTION DOCK
1 Acre
BLACKWALL BASIN
6¾ Acres
SOUTH DOCK BASIN
5¼ Acres
POPLAR
WEST INDIA DOCK
30 Acres
WEST INDIA DOCK
24 Acres
SOUTH DOCK
26¾ Acres
LIMEHOUSE BASIN
1¾ Acres
LIMEHOUSE REACH

Fig. 16.

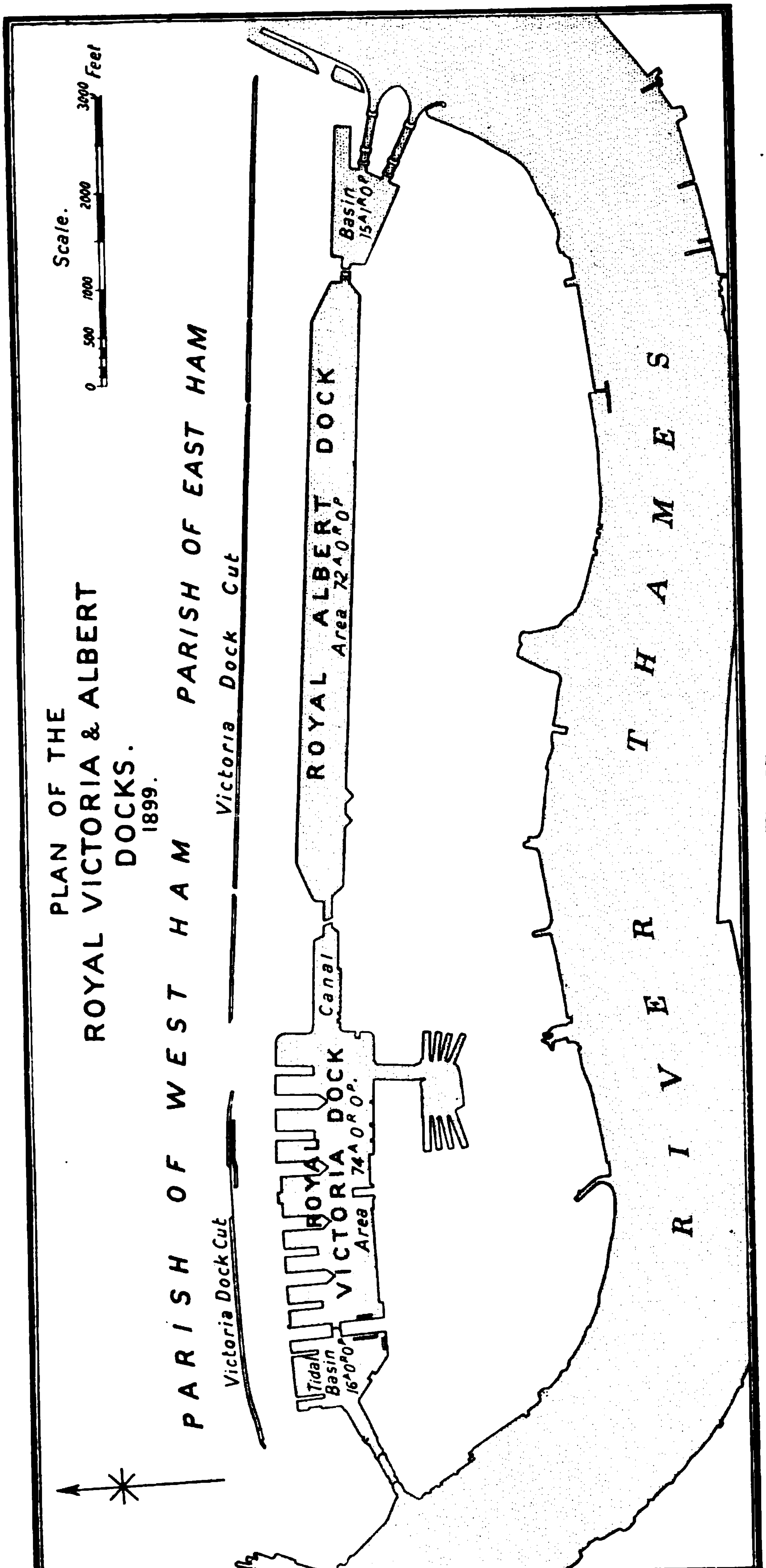

PLAN OF THE
ROYAL VICTORIA & ALBERT
DOCKS.
1899.
Scale.
0 500 1000 2000 3000 Feet
PARISH OF WEST HAM
PARISH OF EAST HAM
Victoria Dock Cut
Victoria Dock Cut
ROYAL ALBERT DOCK
Area 72A 0R 0P
Basin
15A 1R 0P
Canal
ROYAL VICTORIA DOCK
Area 74A 0R 0P.
Tidal Basin
16A 0R 0P
RIVER THAMES

Fig. 17.

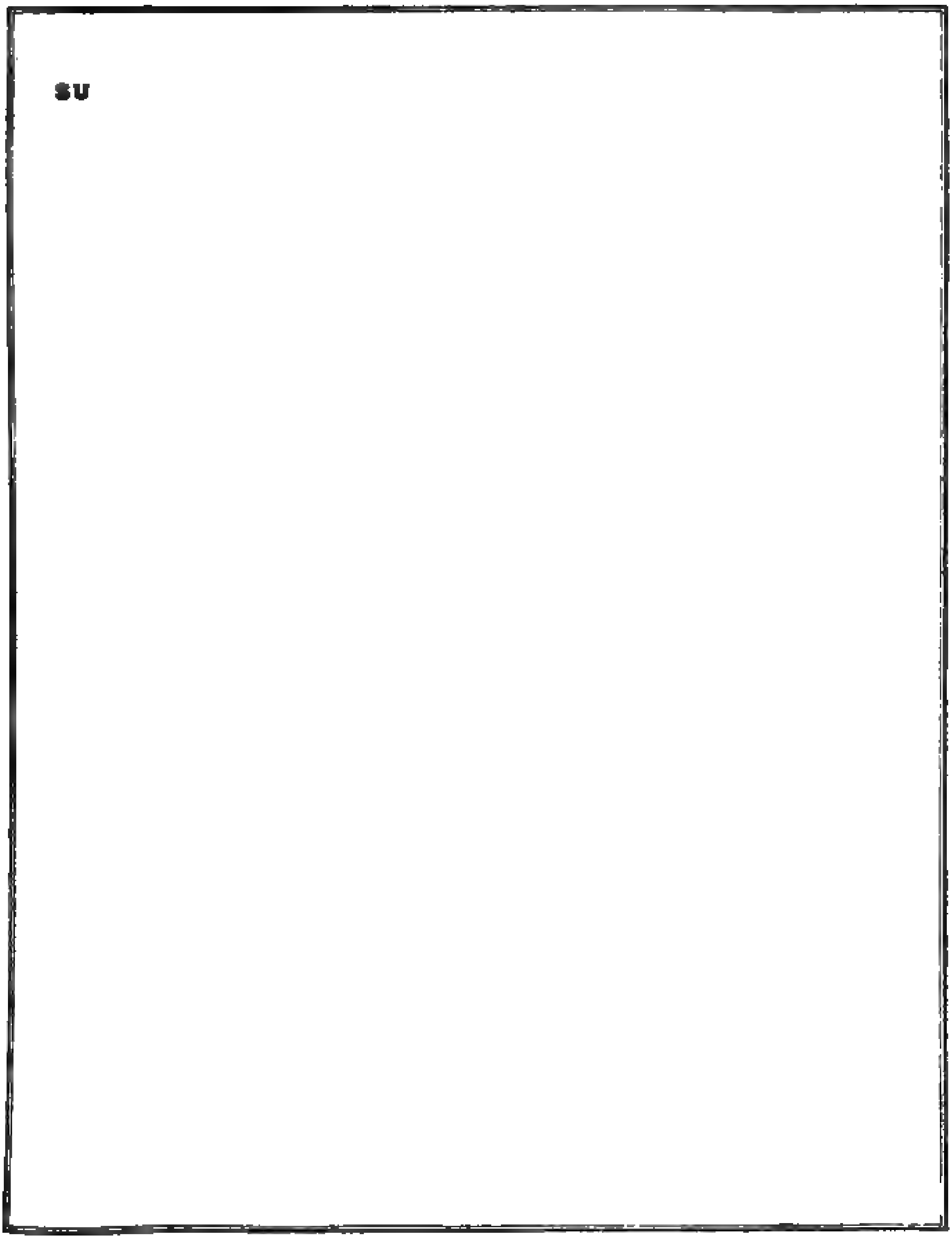


Fig. 18.

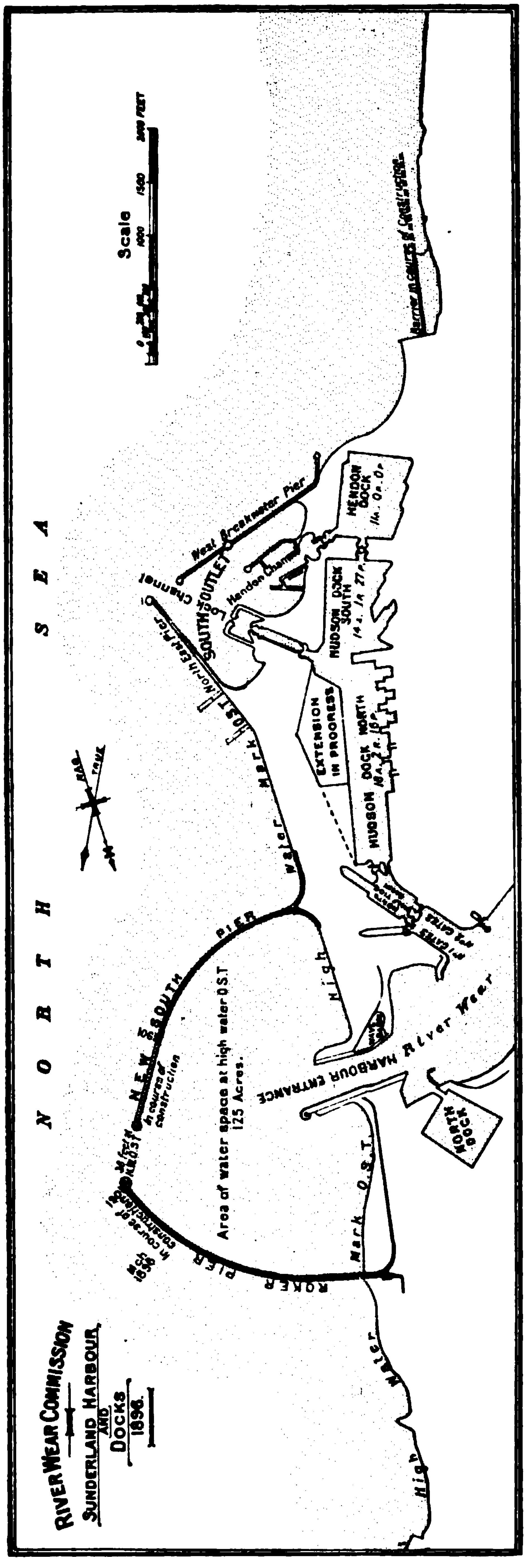


Fig. 19.

The Roker Pier was completed in September, 1903.
The area of the Hudson Docks is in course of being increased by 8½ acres.

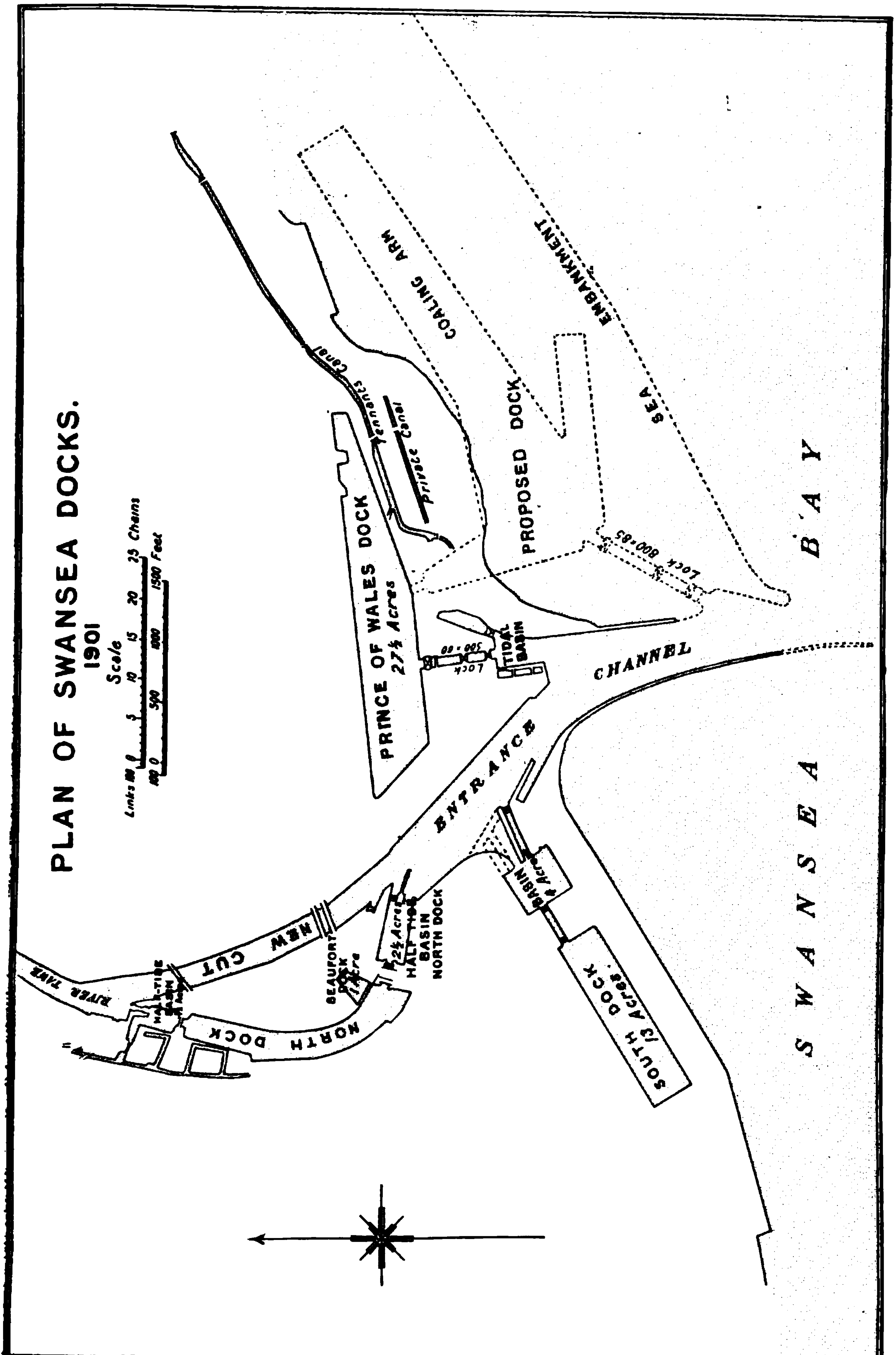

PLAN OF SWANSEA DOCKS.
1901
Scale
Links 100 0 5 10 15 20 25 Chains
100 0 500 1000 1500 Feet
PRINCE OF WALES DOCK
27¼ Acres
Tennants Canal
Private Canal
Canal
COALING ARM
EMBANKMENT
SEA
PROPOSED DOCK
Lock 800×85
TIDAL BASIN
Lock
CHANNEL
ENTRANCE
NEW CUT
RIVER TAWE
HALF-TIDE BASIN
NORTH DOCK
BEAUFORT DOCK 1 Acre
2¼ Acres
HALF-TIDE BASIN NORTH DOCK
BASIN 4 Acres
SOUTH DOCK 13 Acres
SWANSEA BAY

Fig. 20.

Fig. 21.

PLAN OF THE PORT OF HAVRE.

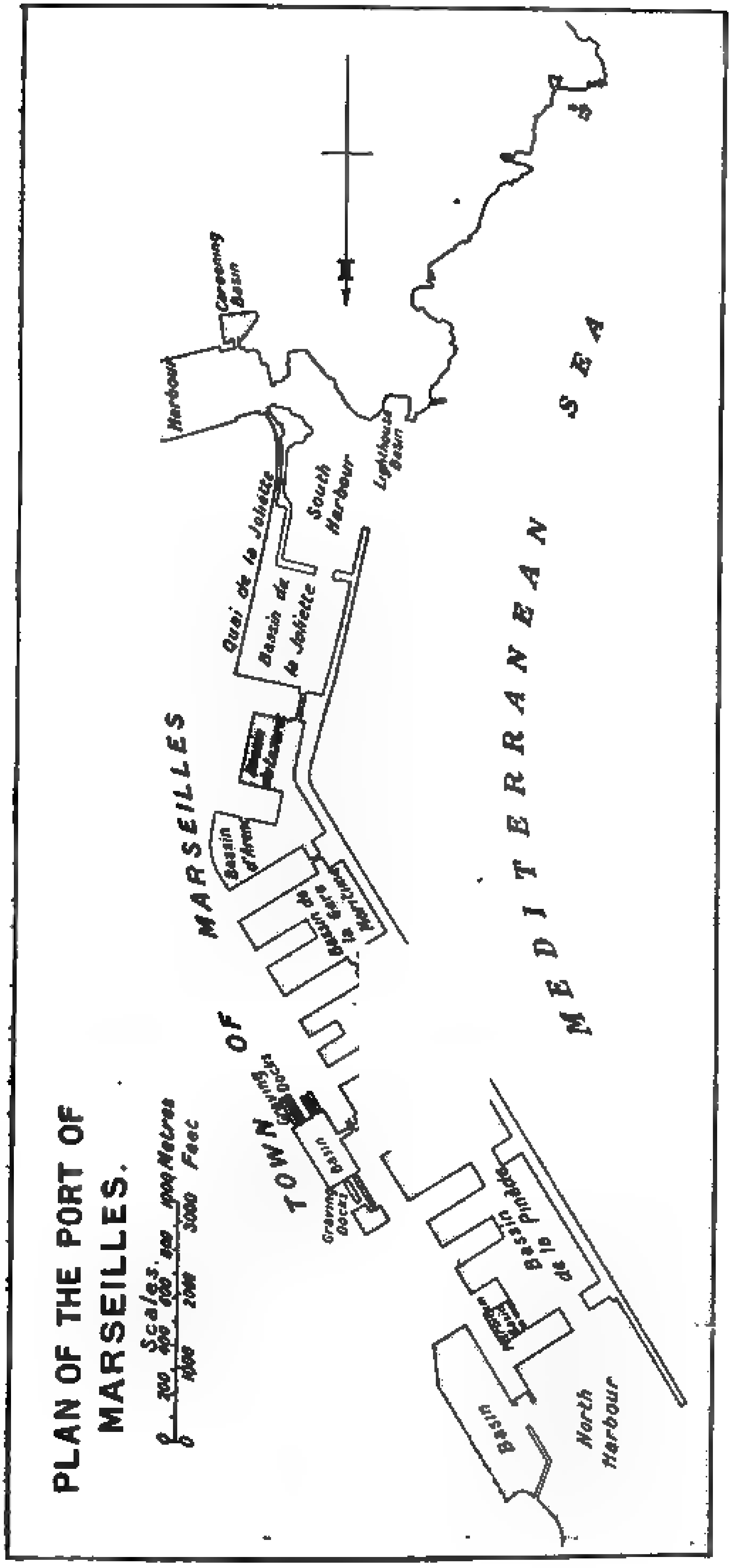


Fig. 22.

The Bassin de la Pinède is at present in course of construction.

TABLE V.—Representative Docks and Basins.

Compiled mainly from Information in Appendix to Lloyd's Register, 1903-1904.

PORT.	Name of Dock or Basin.	Area in Acres.	Length in Feet.	Breadth in Feet.	Quayage and Wharfage in Feet.	Depth of Water on Sill. At H.W.O.S.T.	Depth of Water on Sill. At Highest Springs.	Remarks.
						Feet.	Feet.	
Aberdeen, .	Victoria, . .	33½	2,022	465 (mean)	4,327	...	27¾	
Barrow, . .	Buccleuch, . .	31	3,000	480	5,000	...	...	24 feet depth of water on sill kept practically constant.
,, . .	Devonshire, .	31	2,500	500	5,700	...	...	26 feet ,, ,,
Barry, . .	No. 1, . . .	73	3,100	1,100 (max.)	10,500	37⅜	42	Curved sills; depth of water 3 feet deeper at centre.
,, . .	No. 2, . . .	34	3,320	400-600	7,000	37⅜	42	,, ,, ,,
Belfast, . .	Abercorn Basin, .	10½	725	646	2,410	...	24⅛	2 jetties.
,, . .	York, . . .	10¼	1,581	240	4,142	...	39⅛	
Bristol, . .	Avonmouth, .	19	2,180	500	4,800	38	41	
Cardiff, . .	Bute, East, . .	46¼	4,300	300 and 500	9,360	31¾	35	
,, . .	,, West, .	19½	4,000	200	8,800	28¾	32	
,, . .	Roath, . .	33	2,400	600	7,520	35¾	...	1 jetty 800 × 60 feet.
,, . .	Bute, New, . .	50	2,550	800-1,000	6,700	42¼	...	
Dublin, . .	Alexandra Basin,	50	...	...	2,160	...	...	23 feet deep at low water.
Dundee, . .	Camperdown, .	8½	670	550	2,400	21⅜	23⅛	
,, . .	Victoria, . .	10¾	1,300	380	2,750	21⅜	23⅛	2 jetties.
Glasgow, . .	Queen's, . .	33¾	Irregular.	Irregular.	10,000	...	...	Tidal basin 35 ft. deep at highest springs.
,, . .	Prince's, . .	35	,,	,,	11,210	...	...	,, 37 ft. ,, ,,
Greenock, .	James Watt, .	14¼	2,000	300-350	6,400	32	...	1 jetty 800 feet long.
Great Grimsby,	Alexandra, . .	48	6,300	450	6,800	18	19½	
,, ,,	Royal, . .	25	2,328	500	3,917	26	27½	
Hartlepool, .	Victoria, . .	19	1,935	480	5,320	21½	...	Jetty 310 feet long.
,, (West),	Central, . .	13	990	660	2,585	26	...	
,, . .	Jackson, . .	13	920	500	2,680	21½	...	
Hull, . .	Albert, . .	24½	3,420	300 (mean)	7,100	...	28¼	
,, . .	Alexandra, . .	46½	2,300	1,000	6,200	34	...	4 jetties from 430 to 550 feet long.
,, . .	Victoria, . .	20	2,100	378	4,761	...	26	
Leith, . .	Edinburgh, . .	16⅜	1,500	650	6,000	25½	26½	1 jetty 1,000 feet long.
,, . .	Albert, . .	11	1,100	450	3,000	25½	26½	
,, . .	Imperial, . .	19	1,900	280-550	...	30½	31½	
Liverpool, .	Hornby, . .	17	1,625	425	5,319	31	...	
,, . .	Alexandra, . .	44	Irregular.	Irregular.	11,760	39½	...	Including 3 branch docks.
,, . .	Langton, . .	18	1,175	400-880	5,980	31	...	

TABLE V.—Docks and Basins (*Continued*).

Port.	Name of Dock or Basin.	Area in Acres.	Length in Feet.	Breadth in Feet.	Quayage and Wharfage in Feet.	Depth of Water on Sill.		Remarks.
						At H.W.O.S.T.	At Highest Springs.	
						Feet.	Feet.	
Liverpool, .	Brocklebank, .	11¼	1,420	350	3,000	26¾	...	
,, . .	Huskisson, . .	36½	Irregular.	Irregular.	10,780	39½	...	
Birkenhead, .	West Float, .	52	,,	,,	11,190	26½	...	
,, .	East ,, .	59¾	,,	,,	10,300	26½	...	
London, . .	Tilbury, . .	54	,,	,,	13,000	...	...	
,, . .	Royal Albert, .	72	8,000	490	17,250	...	...	Length includes entrance basin.
,, . .	Royal Victoria, .	74	5,400	1,050	20,500	...	...	,, 14 jetties.
,, . .	Millwall, . .	36	4,300	350	8,800	...	...	
Newcastle, .	Northumberland,	55	3,700	650	450	24	26	1 jetty.
,, . .	Tyne, . .	50	2,490	720	5,340	31⅜	34⅜	
Newport (Mon.),	Alexandra, N., .	28¾	2,500	500	5,681	35	40½	
,, ,, .	,, S., .	24	1,500	550	4,100	35	40½	
Southampton, .	Empress, . .	18½	1,720	810	3,635	35½	41	Tidal basin.
Sunderland, .	Hudson, . .	33	2,800	420	8,710	28	32	6 jetties.
,, .	Hendon, . .	11	800	660	2,800	26½	30½	
Swansea, . .	Prince of Wales, .	27½	3,210	500	6,800	32	35¼	
Antwerp, .	Kattendyk, .	...	3,066	459	5,912	21½	...	
,, . .	Kempisch, . .	...	1,148	492	3,180	23	...	
Dunkirk, . .	Freycinet, . .	55½	...	...	16,370	36	26	
,, . .	Commerce, . .	13½	...	...	2,772	20⅜	22½	
Havre, . .	Vauban, . .	...	2,680	328 (mean)	5,909	20½	...	
,, . .	Eure, . . .	...	3,083	920 ,,	6,726	38	...	
,, . .	Bellot, . .	...	3,100	722 ,,	8,660	35⅓	...	
Marseilles, .	La Joliette, .	54	1,640	1,312	8,148	28	...	
,, .	National, . .	102¼	3,034	1,689	13,181	...	...	Depth of water in dock varies from 19½ to 59 feet.
Rouen, . .	Principal, . .	79	7,546	557 (mean)	13,665	...	...	
St. Nazaire, .	Penhouet, . .	55½	3,609	525-755	9,530	...	30½	
Bremerhaven, .	New Kaiserhafen,	44	...	...	5,544	34	...	
Hamburg, .	Sandthorhafen, .	24¾	3,412	382	6,752	24¾	...	
,, . .	Segelschiffhafen,	85½	4,429	919 (mean)	10,494	27¼	...	
,, . .	Hansahafen, .	90¼	5,348	1,328	7,106	27¼	...	
Rotterdam, .	Maashafen, .	143	...	...	...	...	...	
Calcutta, .	Kidderpur, . .	33	2,600	600	5,400	34	...	
Capetown, .	Alfred, . .	8½	1,010	470	2,400	...	...	

CHAPTER III.

CONSTRUCTIVE APPLIANCES.

Classification—Positive, Negative, and Auxiliary Appliances—Piling Apparatus—Hand, Steam, Electric, and Hydraulic Machines—Ram and Fall—Quiescence—Limit of Driving—Supporting Power of Piles—Concrete Mixers—Messent, Taylor, Carey-Latham, Sutcliffe, and Gravity Machines—Concrete Moulds—Block-setting Appliances—Excavators—French and German Machines—Ruston, Simpson-Porter, and Whitaker Steam Navvies—Hydraulic Navvy—Drilling Appliances—Hand and Machine Drills—Blasting Agents—Haulage and Traction—Dredgers and Hoppers—Suction, Ladder, Dipper, and Grab Dredgers—Buckets—Shoots—Tumblers—Power—Cost—Dams of Earth, Timber, Stone, Concrete, and Iron—Cofferdams—Strength and Stability—Pumps and Pumping—Cranes—Overhead Travellers—Skips—Lewis Bars and Clips—List of Constructive Plant at Keyham Dock Works.

The speediest and, at the same time, the most economical methods of carrying out projected works are points of the utmost importance to the engineer. Time and capital are alike too valuable to be utilised to any but their fullest extent. Hence some consideration of the various types of appliances used in dock construction, especially in regard to their capabilities, efficiency, and cost, will not be without both interest and advantage.

Classification.—A chapter which has to deal with a number of disconnected items must perforce exhibit some break of continuity, and in order to minimise this effect and, at the same time, to link together the different sections, the following classification is proposed:—*Positive appliances* will include all those employed in definite constructive operations—that is to say, piling machines, concrete mixers, setting machines, and the like. *Negative appliances* will be understood to mean those engaged in the removal of existing obstacles and superfluous material, such as excavators and dredgers, boring apparatus and blasting agents. A third class, designated *Auxiliary appliances*, will include all those used in conjunction with each of the foregoing, indifferently and for either object, as dams, pumps, waggons, skips, and locomotives. The classification is, of course, purely artificial, but it will serve a useful purpose if it admits of the systematic treatment of a heterogeneous subject.

POSITIVE APPLIANCES.

Pile Drivers and Driving.—The process of driving a pile generally consists in causing a heavy weight, called a *ram* or *monkey*, to fall from some height, in a series of blows, upon the head of the pile. For this purpose a

piling machine is constructed, with two long vertical guides or runners, up and down the face of which the monkey slides, being kept in position by a lug or projection fitting into the groove between the guides.

The simplest kind of pile driver is the *ringing machine*, in which the work is performed entirely by hand. The monkey rarely weighs more than one-third of a ton, and it is lifted by a rope which, after passing over the pulley at the head of the frame, is connected with a number of short lengths, so as to afford a hold to a corresponding number of men, in the proportion of about 40 lbs. weight per man. The lift does not exceed 4 feet. At a given signal the monkey is allowed to fall, the men taking advantage of each rebound to raise the monkey. Driving is usually carried on in spells of three or four minutes' duration, with intervals of rest, and in this way men are said to be capable of delivering from 4,000 to 5,000 blows per day.

The explosion of a cartridge has been utilised to augment the effect of the blow upon the pile and to increase the recoil of the ram. The cartridges are inserted in a small hollow in the pile-head, and, after percussion, are replaced by fresh ones during the ascent of the ram.* The cartridges are either of gunpowder or of dynamite; in the latter case the head of the pile is protected by an iron plate. Explosive drivers can readily make from 30 to 40 blows of from 5 to 10 feet per minute.

In another pile driver, called a *crab engine*, the rope, instead of being directly held by hand, passes round the drum of a crab or windlass, by means of which the monkey can be given a fall of 10 or 12 feet.

Such merely manual methods, however, are primitive; they are really only suitable for driving small piles in insignificant numbers, and are entirely superseded in works of importance by steam piling machines.

Steam Piling Machines are of various design. In the earlier examples steam power simply replaced manual effort in lifting the weight. A hook or trigger at the end of the lifting chain engaged in a staple in the head of the pile, and was released by pulling a counter-weighted lever. The chain had to be lowered after each blow.

The intermittent action involved in this arrangement has been obviated by the device of an endless lifting chain, characteristic of the machines of Messrs. Sissons & White (fig. 23). The chain passes up the groove between the leaders and down over a spur wheel in the gearing, by means of which it is kept running continuously. The ram, which weighs from 15 to 25 cwts., is raised by a tongue passing through its centre and capable of engaging in the moving chain, through the medium of a rack and pinion movement actuated by a lever. The man in charge of this apparatus pulls the cord attached to the end of the lever, causing the tongue to shoot out at the back of the monkey into the nearest open link. The tongue is withdrawn at any desired level by the other end of the lever coming in contact with a staple fixed to the face of one of the guides. The holes for these

* Sir F. Bramwell, Presidential Address, 1885, *Min. Proc. Inst., C.E.*, vol. lxxx.

staples are set at short intervals, so that the amount of fall can be regulated fairly uniformly.

The services of three men are required for each machine—one to work the winch, another to actuate the lever, and a third to watch the pile, mark its progress, and shift the staple. Steam should be partially cut off while the ram is falling, in order to reduce the speed of the chain for re-attachment.

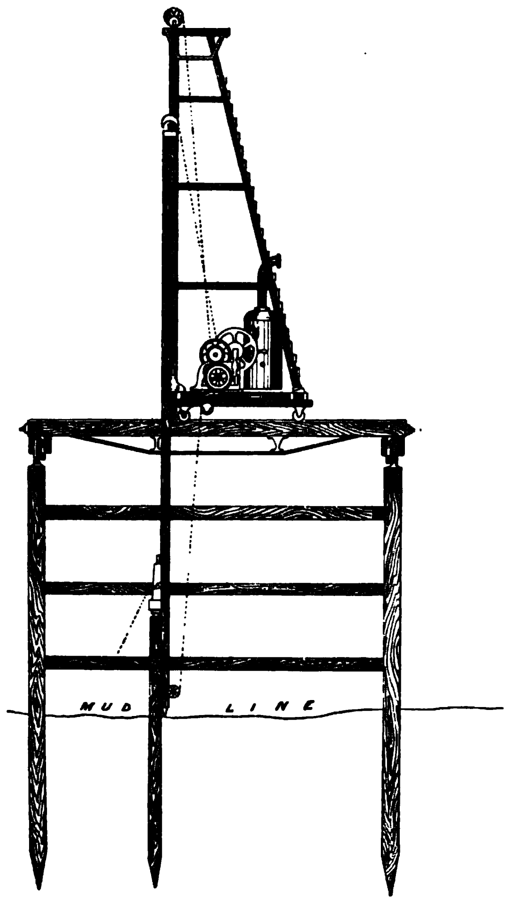


Fig. 23.—Pile-driving Machine fitted with Telescopic Leaders.

The pile is pitched by an auxiliary chain passing over a separate pulley at the frame head. It is kept in position by a toggle bolt passing right through the pile near the top, and also through a wood block in the groove behind it, to the back of the leaders, where it is secured by an iron bearing plate, nut, and screw. After the necessary preliminaries the monkey, which has been temporarily raised out of the way, is lowered upon the pile and is ready for action. The rate of working is about six blows per minute under steam pressure of 55 to 60 lbs. per square inch. Piles can be driven vertically, or at any required inclination, by adjusting a screw at the foot of the ladder at the rear of the platform.

When the pile has been driven below the level of the foot of the leaders, the process is continued, either by the interposition of a *punch* or *dolly* (a short log of the same scantling as the pile) between the pile and the ram, or by the use of telescopic leaders (fig. 23). The first method involves considerable loss of driving power, as the dolly absorbs fully one-half of the kinetic energy of the blow.

The frame of a *Whitaker steam-hammer pile driver* is not dissimilar from that just described. The principal difference of the contrivance lies in the application of the power. Driving is done by means of a piston and cylinder,

Fig. 24.—Section of Whitaker's Steam-hammer Pile Driver.

but the action is the reverse of ordinary usage. The piston (fig. 24) is kept stationary and in continuous contact with the pile head, while the blow is administered by the lower end of the heavy cast-iron cylinder, moving up and down under steam pressure. The movement of the cylinder is guided by rollers behind the main leaders, and the arrangement involves a sliding steam feed pipe (which is a special feature of the system), with a flexible rubber connection to the supply pipe from boiler. At the head of the cylinder is a two-way cock, regulated by a double-armed lever, which, when pulled down on one side, exhausts the cylinder, and on the other admits fresh steam. There is also a double-action machine, in which steam pressure is applied alternately to each side of the piston, thus increasing the force of the blow. The stroke is about 3 feet, and blows can follow one another with great rapidity. From observations of a Whitaker machine in single action with 80 to 90 lbs. steam pressure, the author finds that 35 blows per minute can be delivered at full stroke, or 60 blows per minute with a stroke of 12 inches. In double action 45 blows per minute were delivered with a stroke of 2 feet. The weight of the cylinder was 1 ton.

A similar machine, known as the *Cram Pile Driver*, manufactured in America, has a hammer fastened to the lower end of the cylinder, and is supplied with steam through a hollow piston-rod. The original *Nasmyth hammer* is also used, in which the hammer is attached to the piston, the cylinder remaining stationary and being confined between the upper ends of two vertical and parallel I or channel beams, the lower ends of which enclose a hollow, conical bonnet casting, fitting over the head of the pile. This casting is open at the top, and through it the blow is administered. When steam is admitted to the cylinder, the hammer is lifted about 30 to 40 inches and then allowed to fall, generally by the automatic opening of an escape valve.

Piling machines of the steam-hammer type consume from 1 to 2 tons of coal per day, working with a boiler pressure of 50 to 75 lbs. per square inch, and can deliver blows at the rate of about 60 per minute. They need three men in attendance.

The disadvantage attending them is the liability of the pilehead to crushing or *brooming*, which, combined with the escape of moisture from the cylinder, reduces it, if in the least degree soft or sappy, to a saponaceous condition. The effect of this is to materially diminish the force of the blow, as is evidenced by the following particulars of the driving of a green Norway pile by a Nasmyth steam hammer * :—

The 3rd foot of penetration required	5 blows.
,, 4th ,, ,, ,,	15 ,,
,, 5th ,, ,, ,,	20 ,,
,, 10th ,, ,, ,,	73 ,,
,, 12th ,, ,, ,,	153 ,,
,, 14th ,, ,, ,,	684 ,,

* Whittemore on "The Efficiency of Pile Driving," *Min. Proc. Inst. C.E.*, vol. lxxvi., p. 399.

HEAD ADZED OFF.

The 15th foot of penetration required	213 blows.
,, 18th ,, ,, ,, ...	825 ,,

HEAD SAWN OFF.

The 19th foot of penetration required	213 blows.
,, 22nd ,, ,, ,, ...	378 ,,

The total number of blows was 5,228. A similar pile, which was not adzed or sawn, required 9,923 blows to descend to the same depth. The ram weighed 2,800 lbs. and fell 3 feet sixty-five times per minute. The friction caused by the working of the fibres on each other, under the blows of the hammer, was sufficient to ignite and burn the interior of the head of the pile from side to side.

A third type of pile driver is the *Electric Pile Driver*, in which advantage is taken of the temporary magnetisation of wrought iron to make an electro-magnet of that material attach itself by contact to the cast-iron monkey. The two parts are then lifted by the winch. On switching off the current the monkey falls, and the magnet is caused to follow it down ready for lifting again. The monkey is of the ordinary kind, with an upper planed surface. The magnet is connected by wires to the motor on the winch. The illustration (fig. 25) is of one manufactured by the New Southgate Engineering Co., Ltd.

Hydraulic Method.—While piles readily respond to the motive force of the ram in ordinary ground, and even in stiff clay, their progress through sand and gravel is not so satisfactory, and the ordinary methods of driving have generally to be abandoned, either wholly or partially, in favour of the water jet. The principle of this method consists in transforming the sand immediately beneath the pile into quicksand, by saturating it with water under pressure, a condition which enables the pile to sink by its own weight or with very little assistance. The water is conducted to the foot of the pile by means of wrought iron gas piping having a short returned end, provided with a nozzle or pierced with holes, which passes underneath the pile. This last is not usually pointed, but left with a butt end, which favours perpendicularity in driving. The descent of the pile may be expedited by a static weight, or by the direct downward pull of a rope passing through sheaves to a winch. When the pile has been sunk to a sufficient depth, the nozzle of the water pipe is turned through a quadrant to clear the pile and brought up to the surface again by the same means which accomplished its descent. The sand is then allowed to consolidate round the pile, which it does rapidly and satisfactorily. No difficulty is experienced from boulders or large stones for, if met with, they can be displaced or lowered by a preliminary action of the jet below them.

This hydraulic method of sinking piles is often used in conjunction with the falling ram in earth of a compact nature. The pile in this case is naturally furnished with a pointed end, preferably conical.

Timber piles are universally in evidence, but iron and concrete piles also

Fig. 25.—Electric Pile Driver.

have their uses. The drawback to timber piles is that, although extremely durable while completely protected from atmospheric influence, they are very susceptible to decay in air and, more particularly, "betwixt wind and water," and to perish from the attacks of insects.

Iron piles with pointed ends, and concrete piles on the Hennebique system (figs. 26 and 27), (*vide* also Chap. vii.) should only be driven through the interposition of a wooden dolly (fig. 28).

For untrustworthy strata of indefinite depth, piles, whether of wood or iron, are occasionally furnished with a broad screw end to the extent of a single turn or slightly more. This considerably increases the bearing surface. Such piles have to be lowered by rotation, either by means of manual, animal, or mechanical power

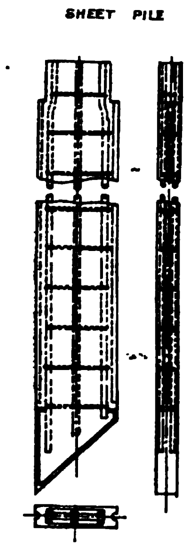


Fig. 26.—Hennebique Pile.

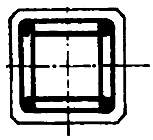


Fig. 27.—Bearing Pile.

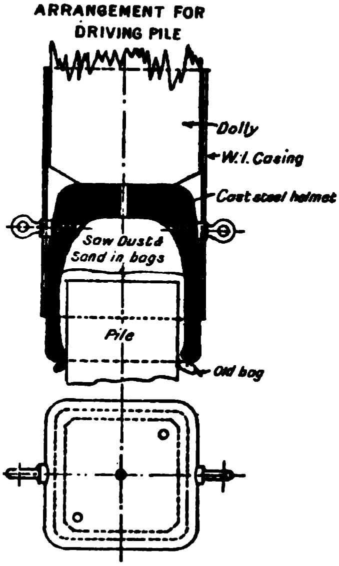


Fig. 28.—Hennebique Pile.

Ram and Fall.—Piles may theoretically be driven at the same rate with a light ram and a long fall as with a heavy ram and short fall, but the second method is preferable in practice. A long fall means greater oscillation in the ram and a consequent jar in the delivery of the blow, which tends to rupture the pile. From extensive experience in the driving of wooden piles, the author finds a monkey of 1 ton weight, with a fall of 8 or 10 feet, a very suitable combination. For concrete piles on the Hennebique system, even less fall is desirable, and a monkey of $2\frac{1}{2}$ tons, with $4\frac{1}{2}$ feet drop, has answered very satisfactorily at Southampton.

Quiescence.—If the driving of a pile be interrupted for a short time, it is found that the resistance offered to driving is materially increased. Piles which have been left partially driven overnight have exhibited a resistance nearly three times as great on the resumption of work in the morning. This

effect is no doubt due to the consolidation round the pile of the earth which had been maintained in a state of disintegration and vibration during a sequence of rapid blows.

Limit of Driving.—The limit of adequate driving and the maximum supporting power of piles are equally moot points among engineers. To a certain extent they are interdependent.

The practice at Liverpool has been to regard a total depression, not exceeding $\frac{1}{4}$ inch in 10 blows of a 20-cwt. ram falling 10 feet, as evidence of sufficient driving, or, in other words, an expenditure of mechanical energy at the rate of 896,000 ft.-lbs. per inch. At New York river wall the piles were specified not to penetrate more than $\frac{1}{10}$ foot with the last blow of a 3,000-lb. monkey falling through 8 feet, involving energy to the extent of 20,000 ft.-lbs. per inch. According to Rankine, some of the best authorities consider the test of a sufficiently driven pile to be a depression of not more than $\frac{1}{5}$ inch by 30 blows of an 800-lb. ram falling 5 feet, or mechanical energy represented by 600,000 ft.-lbs. per inch. These standards are evidence of the great diversity of opinion there is on the subject.

Supporting Power.—Various theoretical and empirical formulæ have been suggested for determining the relationship between the blow required to drive a pile to a given depth and the greatest load it will sustain without sinking further.

Rankine * puts forward the following equation, supposing the pile to be supported by uniformly distributed friction against its sides :—

$$p = \sqrt{\left(\frac{4\,\mathrm{E}s w \mathrm{H}}{\mathrm{L}} + \frac{4\,\mathrm{E}^2 s^2 \mathrm{D}^2}{\mathrm{L}^2}\right)} - \frac{2\,\mathrm{E}s\mathrm{D}}{\mathrm{L}} \quad . \quad . \quad (1)$$

in which

w = Weight of ram in lbs.
E = Modulus of elasticity.
s = Sectional area of pile in square inches.
H = Fall of ram in feet.
L = Length of pile in feet.
p = Maximum load in lbs.
D = Depression of the pile in feet by the last blow.

A factor of safety of not less than 3 should be used; preferably one of 5 to 10.

A very well-known, but merely approximate, rule devised by Major Saunders of the U.S. Engineers is

$$f = \frac{w\mathrm{H}}{8\,\mathrm{D}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

f being the safe load in lbs. and the other notation as before.

The formula recommended by Trautwine is

$$p = \frac{51{\cdot}5\, w \sqrt[3]{\mathrm{H}}}{12\,\mathrm{D} + 1} \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

with a factor of safety of from 2 to 12 according to circumstances.

* *Manual of Civil Engineering*, p. 604.

The majority of the formulæ enunciated for dealing with the question of the supporting power of piles are of a very complicated nature, and comprise elements which are but remotely connected with it. Mr. C. H. Haswell has the following pertinent remarks upon the subject * :—

"The resistance opposed by a pile to the blow of a ram is the measure of its value to sustain stress whatever may be its diameter, weight, length, or modulus of elasticity. The diameter and length of a pile do not affect the question, their effect is to limit penetration. The weight of the pile is worthy of consideration only as affecting the weight of the ram employed. The relative elasticity is of little moment, for when a pile approaches the limit of its penetration its head is dressed off, if broomed, and if split or liable to be so, it is confined by a ring. In fact, the weight of the ram being proportioned to the duty required of it, the diameter, length, and elasticity of the pile are inconsiderable, where so great factors of safety ranging, in various formulæ, from $\frac{1}{12}$ to $\frac{1}{3}$, are employed."

Mr. Haswell's own formula is

$$f = \frac{32\, w \sqrt{H}}{C} \quad . \quad . \quad . \quad . \quad . \quad (4)$$

in which the constant (C) has values ranging between 3 and 6, according to the nature and condition of the soil, the character of the piles, and the excellence of their driving.

The following table exhibits a comparative view of the results obtained by the foregoing expressions, assuming a depression of, say, $\frac{1}{2}$ inch from the final blow of the ram in each case. Sectional area of pile = 100 square inches :—

	Safe Load in Lbs.			
	Rankine.	Saunders.	Trautwine.	Haswell.
1,000 lbs. falling 20 feet, . .	66,425	60,000	{ 46,609 7,768 }	{ 47,680 23,840 }
2,000 lbs. falling 25 feet, . .	129,249	150,000	{ 100,296 16,716 }	{ 106,666 53,333 }
3,500 lbs. falling 9 feet, . .	93,312	94,500	{ 124,974 20,829 }	{ 112,000 56,000 }

Rankine's empirical rule for the safe load on a pile, driven till it reaches firm ground, is 1,000 lbs. per square inch of area of head. The author considers 10 cwts. per square inch well within the limit of practical safety.

When the arrangement of the strata is such that it is impossible to reach firm ground with a pile, the conditions of equilibrium are different. The pile will then only be able to sustain a superimposed weight by reason of the friction of the ground against its sides. Under such circumstances Rankine recommends 200 lbs. per square inch as the maximum load. Mr. Hurtzig

* Haswell on "Formulas for Pile Driving," *Min. Proc. Inst. C.E.*, vol. cxv.

gives the following equation,* from experience gained in drawing a number of piles against the frictional resistance of clay.

$$D = \frac{WH}{P} - \frac{P}{500} \quad . \quad . \quad . \quad . \quad . \quad (5)$$

the weight of the ram (W) being taken in tons. P is the extreme resistance of the pile, also in tons. H and D, as before, are the height of fall and the depression under the last blow respectively, both in feet.

Concrete Mixers.—Concrete can be very efficiently mixed by hand, but the process is slow and only suitable in dealing with small quantities. When the requirements are large, as in block and mass work, it will be much more economical and expeditious to employ mechanical agency.

So many varieties of concrete mixers, each with its own particular merits, are on the market, that it is an utter impossibility within the limits of a moderate chapter to review them all; and although it is a somewhat invidious task to select one or two examples for illustration, such a step is inevitable, and must not be understood to convey any depreciation of those machines which are unavoidably excluded.

The principal features of an efficient concrete mixer are a thorough and intimate incorporation of the ingredients and a rapid and regular discharge of material.

Concrete mixers are of two kinds—intermittent and continuous. In the former class, charges are mixed separately; in the latter, they follow one another in unbroken sequence. More perfect incorporation of the ingredients is the particular claim of the intermittent mixers, while the continuous mixers afford greater regularity of supply. In both instances, that machine must be reckoned best in which the churning action is most thorough.

Intermittent Mixers—*Messent Mixer.*—The best known of the earlier types of mixer is that due to the late Mr. P. J. Messent, of Tynemouth, and the following description of it is extracted from the circular of the makers, Messrs. Stothert & Pitt, of Bath:—

"It consists of a closed box or chamber, A (fig. 29), revolving on an axle, and of such a form as, when half-filled with the materials, to cause them to be turned over sideways, as well as endways, four times in each revolution of the chamber, so that in from six to twelve revolutions (the number necessary being varied according to the weight and nature of the materials) a more perfect mixture is effected than could possibly be produced by hand, or (except in a much longer time) by any other machine.

"For filling concrete into a trench, or the hearting of a pier, the machine is supported over the opening, on two balks of timber; a waggon containing the gravel (and cement in bags) follows on the same line. The hopper, shown in the figure, suspended from a davit, is made to contain the

* Hurtzig on "The Friction of Timber Piles in Clay," *Min. Proc. Inst., C.E.*, vol. lxiv.

proper measure of gravel for a charge, whilst the bags contain the proper quantity of cement, and a cistern near at hand (filled by a flexible hose) the proper quantity of water. Two men standing on the waggon (the sides of which are generally raised so that it contains about twice the quantity of an ordinary earth waggon) are able to fill the hopper in the time employed by four men to give the mixer the requisite number of turns. For counting these a tell-tale is provided, which indicates when the

Fig. 29.—Messent Concrete Mixer.

proper number of turns is completed; the mixer is then stopped with the door downwards. The door fastening is released and the charge of concrete falls into its place, the discharge being instantaneous. The opening of the mixer is then turned upwards, as in the figure, the door is opened (through the dotted arc as shown), the hopper, suspended from the davit, is brought over the opening and at once discharged into it, and the water is run in from the cistern at the same time. The door, which closes water-tight, is then

shut and the mixing resumed, the hopper being meanwhile refilled for the next charge.

"With the hand mixer, above described, a gang of six men, with a boy for attending to the water cistern, can make from 30 to 40 cubic yards of concrete blocks, and a larger quantity of concrete in bulk in a trench in a day, of better quality and at a cheaper rate than can be done by shovel mixing, and when the mixers are turned by steam, twice the above quantities are made." The usual standard sizes have capacities of ½ and 1 cubic yard.

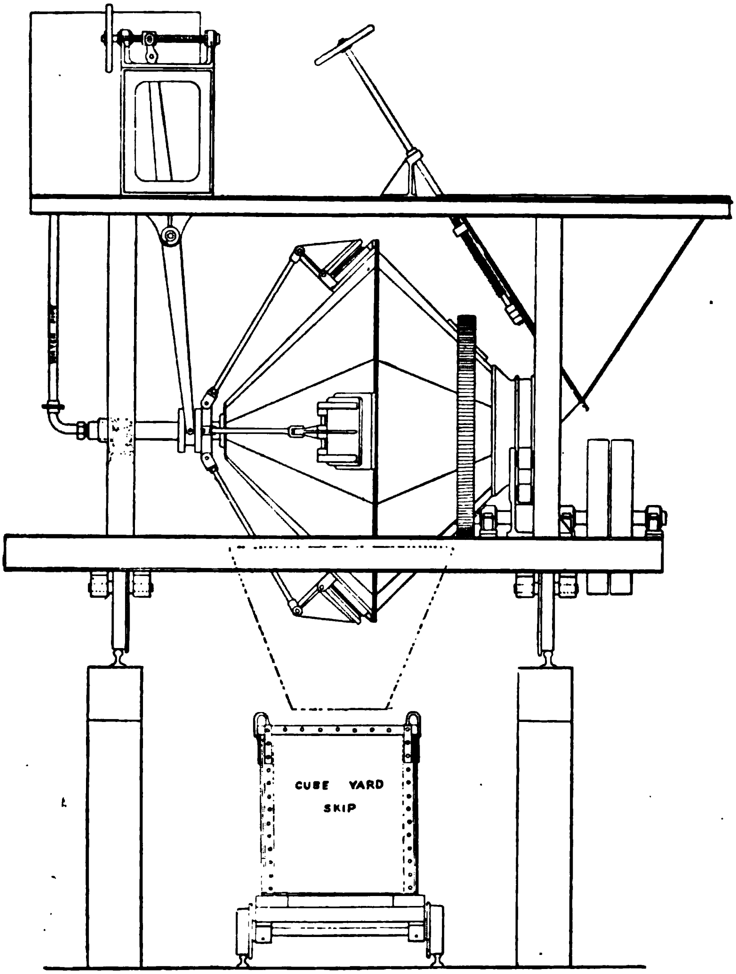


Fig. 30.—Taylor Concrete Mixer.

The Messent mixer has been used at Tynemouth breakwater; Aberdeen harbour works; the Surrey Commercial Docks, London; dock works at Kurrachee, and elsewhere.

Taylor's Mixer.—A later adaptation of the same type of mixer has the revolving chamber in the form of a double cone (fig. 30).

"In mixing concrete the materials are filled into the measuring hopper in the usual proportions; the sliding door is then withdrawn and they are admitted into the revolving mixing cones, to mix dry first; water is then supplied through the hollow supporting shaft.

"A few revolutions, say 15, serve to thoroughly mix the materials, and the delivery doors, which are closed perfectly tight while the mixing is proceeding, being simultaneously opened by the lever and clutch, the contents are dropped through a shoot into suitable trucks or skips, or directly on to the work in hand.

"A one-yard mixer can produce, in ordinary working, 24 cube yards of the very best concrete in one hour at a cost for labour of about 4d. per yard. If fitted with a steam hoist and special service trucks an output of 30 yards per hour of thoroughly well mixed concrete can be guaranteed, the cost being greatly reduced. The machines are made either stationary or portable, and of capacities varying from ⅓ to 1½ cube yards" (*Manufacturers' Circular*).

The machines can also be worked by gas engine or electric motor. They are supplied by Messrs. Henry Balfour & Co., Ltd., of Leven, Fife.

The Taylor mixer has been employed at the Keyham Dockyard extension works, at Barry Dock, at Methil Dock extension, at Seaham Harbour, &c.

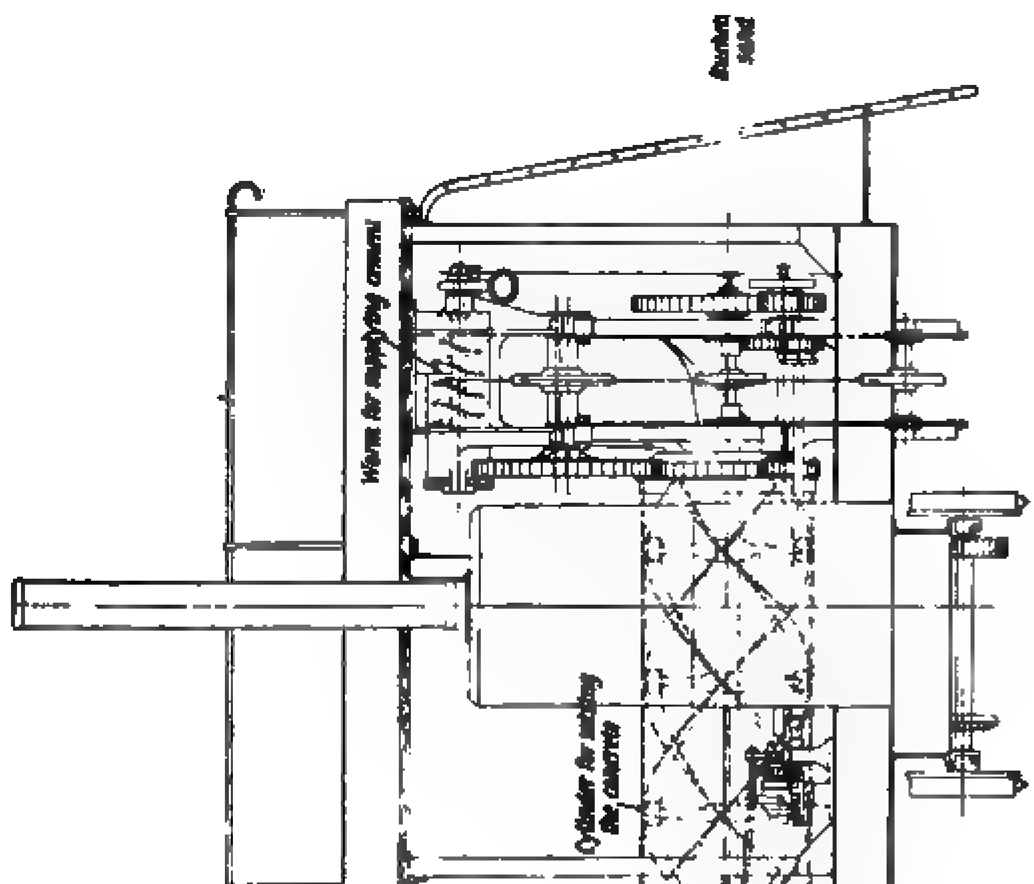
Continuous Mixers—*Carey-Latham Mixer.*—In this machine the sand and ballast are supplied systematically, by means of ascending buckets, to the mixing cylinder (figs. 31 and 32), where they are met by a supply of cement, the quantity of which is regulated by an archimedean screw. The process can thus be carried on uninterruptedly for any length of time.

Incorporation is "carried out in a revolving cylinder in which are fitted inclined blades or vanes, which lift and tumble the materials some 50 times before delivery, first in the dry and afterwards in the wet state. During this process the blades or vanes, which are carried from a central shaft, revolve with the cylinder in the same direction, but at a slightly less speed, whereby they are constantly changing their position, acting as scrapers, and thus prevent the setting of the cement on the blades and inner surface of the mixing cylinder. The water required for the concrete passes through the central shaft, and is sprayed out on the materials as they are tumbled about in the mixer."

The machines are manufactured by Messrs. John H. Wilson & Co., Ltd., of Liverpool, in sizes capable of discharging from 10 to 30 cubic yards per hour.

The Carey-Latham mixer has been used in connection with dock and harbour works at Peterhead, Newhaven, Sydney, Hong Kong, Yokohama, Odessa, Bilbao, New York, &c.

Side Elevation.



End Elevation.

Figs. 31 and 32.—Carey-Latham Concrete Mixer.

Sutcliffe Mixer.—The principle of this machine (fig. 33) is embodied in the method adopted for measuring the quantities of material. The cement is discharged into the uppermost hopper (fig. 34), the floor of which is a cylinder with three grooves of equal area and capacity in its surface. The cylinder is turned by a hand wheel, and an angular displacement of 60° causes the contents of one of the grooves to be emptied into the lower hopper where it meets with the proper supply of gravel. The gravel is discharged from hand barrows, and the cement grooves are so regulated that one grooveful corresponds to a barrow load. When the lower hopper is full, the contents are allowed to fall through three trap doors, opened consecutively, on to a moving band which conveys the dry materials to a

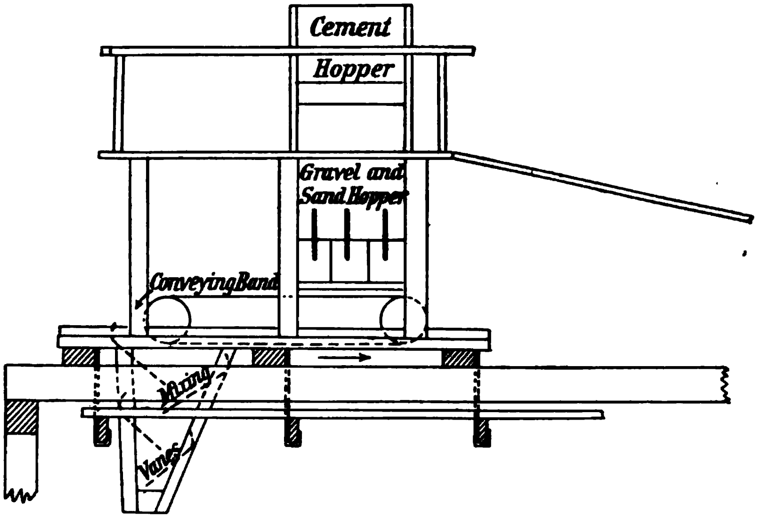


Fig. 33.—Sutcliffe Concrete Mixer—Elevation.

series of trays at the top of a shoot, water being added from a supply pipe at the level of the topmost tray. Each side of the machine is symmetrical, and, by means of an oscillating vane for the deflection of the cement supply, the machine becomes double acting, so that there is absolutely no break in the discharge, which takes place from each side of the lower hopper alternately.

This machine has been very extensively used at the Liverpool Docks for a number of years. It has proved capable of turning out over 300 cubic yards of concrete in a working day of ten hours, but the normal rate of supply lies between 200 and 300 cubic yards per day.

Gravity Mixer.—An American machine in which there are no moving parts, the whole process of mixing being performed by numerous rows of pins, which intercept and sift the material during its descent through a shoot, is effectively illustrated in fig. 35. The ingredients are first deposited in measured quantities upon the platform, and then shovelled by hand to

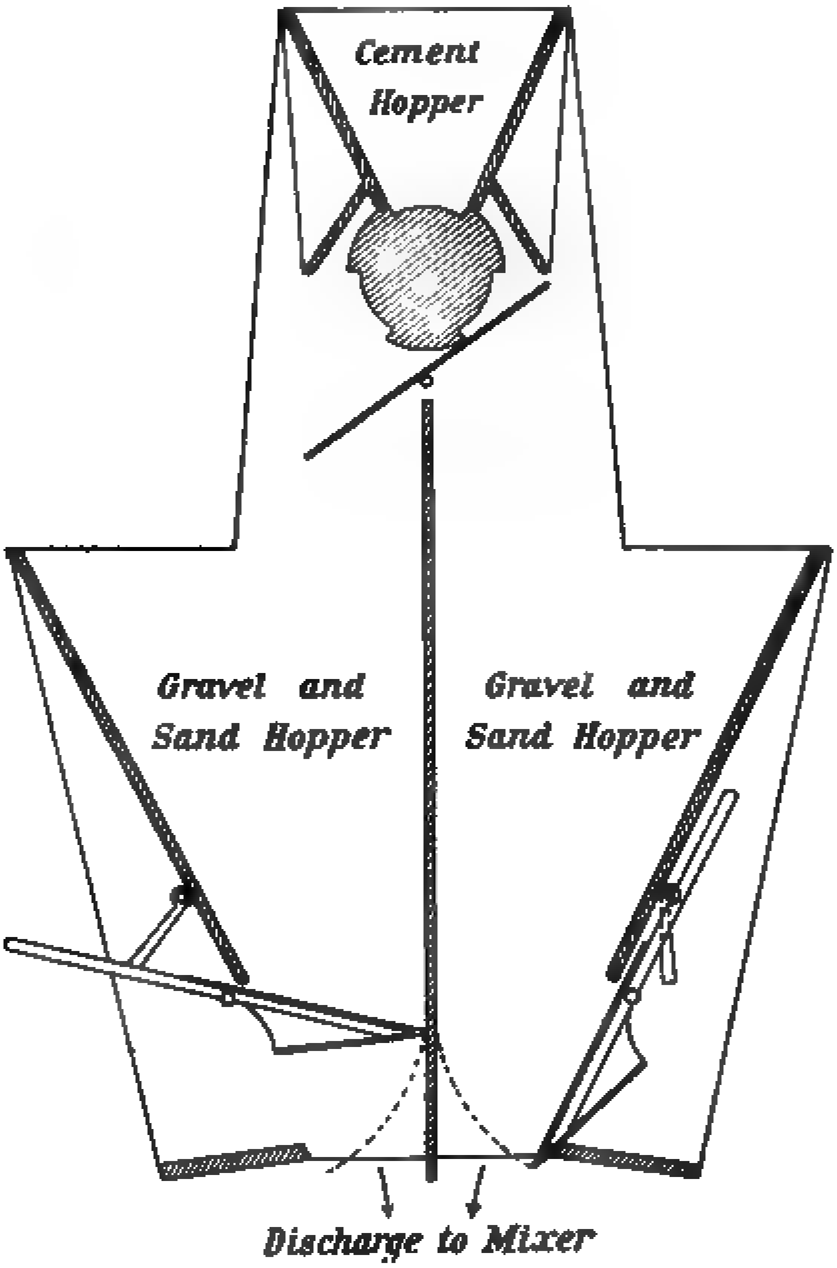


Fig. 34.—Sutcliffe Concrete Mixer—Section of Hoppers.

Fig. 35.—The Gravity Concrete Mixer.

the shoot, at the top of which the water supply is added. The concrete, however, does not actually become wetted until it reaches the fifth row of pins, the process prior to this being that of dry mixing. A gate, or valve, at the foot of the shoot, regulates the discharge if not required to be continuous.

The machine has been used in England at the Liverpool Docks, the London and India Docks, and at Chatham Dockyard. At the Canada Branch Dock (No. 2 contract), Liverpool, it proved capable of discharging rather less than 100 cubic yards per day when fed continuously. This represents a much more moderate output than those of the machines previously described, but the concrete was of a very satisfactory quality. The advantages possessed by a mixer of this type, when used in undertakings not necessitating a rapid supply, are lightness, mobility, and economy.

Concrete Moulds.—Closely connected with concrete mixers are the temporary wooden moulds within which the fluid concrete is deposited.

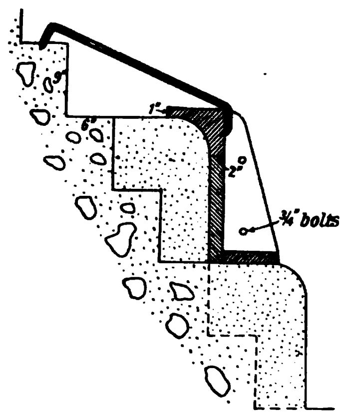


Fig. 36.—Concrete Mould: Section.

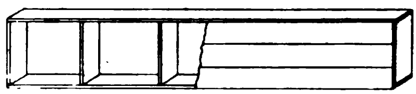


Fig. 37.—Concrete Mould.

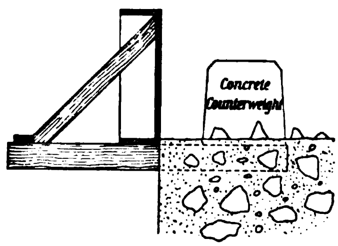


Fig. 38.—Section through Concrete Mould in position.

For the back of a dock or quay wall, which is usually designed in a series of horizontal offsets, ordinary deals on edge form a sufficient and satisfactory enclosure. The offsets are rarely more than a few planks in height; these are stiffened by short verticals at the back, and the whole rests upon the ledge next below. The same method, with a slight modification, may be adopted for the altar courses of a graving dock. The profile is rounded and the face of the mould carefully planed (fig. 36).

For the front of a dock or quay wall, the moulds are prepared in uniform sizes of any convenient dimensions. Two methods of supporting them are illustrated.

In the first case the moulds (fig. 37) rest upon short cantilevers projecting from the wall. These cantilevers (fig. 38) are of timber, about 4 by 3 inches section, with their ends laid upon the previously completed work

and there built in, the whole being carefully levelled. After the wall is completed the cantilever pieces are sawn off, and if the appearance of the ends be deemed unsightly, they are cut out of the wall for an inch or two and the face floated over.

In the second method (fig. 39) long timber uprights are arranged at regular intervals. At each side of the uprights is a groove, within which a mould can slide up or down as required. When raised to each fresh position, it is temporarily secured by wooden wedges.

In every case the surface of a concrete mould should be coated with a suitable oil, or greasy preparation, to minimise adhesion.

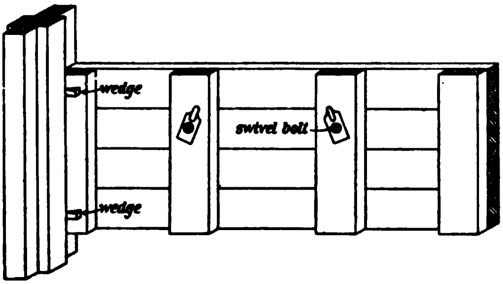


Fig. 39.—Concrete Mould supported by Standards.

Block-setting Appliances.—For the purpose of dealing with massive concrete blocks, used for construction in exposed situations, two types of appliances are generally employed, known by the generic titles of *Goliath* and *Titan* respectively.

The *Goliath* is an overhead traveller, with rectilinear motions. The frame, consisting of two vertical sides connected by an upper horizontal platform, travels backwards and forwards upon two lines of rails at the ground level. The platform supports a track for the transverse motion of the hoisting machine. The general function of a Goliath is the removal of the blocks from the moulds, in which they have been formed, to the stacking yard. An illustration of one is given in fig. 40, which represents a 42-ton steam Goliath, constructed by Messrs. Ransomes & Rapier, of Ipswich, for harbour work at Dover. The span of the main girders, which are 138 feet over all, is 100 feet 1½ inches between centres of tracks, and the clear headway is 25 feet, while the total lift is 120 feet. The speeds of the various movements are :—Lifting, 10 feet; crab travel, 50 feet; main travel, 60 feet per minute. The weight of the machine in working order is 216 tons.*

The *Titan* is also an overhead traveller, but on the cantilever principle. which admits of rotary as well as rectilinear motion. Its function is to take the blocks from the yard and deposit them in their places. The earlier type of Titan did not possess the turning movement, but this latter is very useful in setting apron blocks alongside the main work. The Mormugao

* *Engineering*, September 29, 1899.

Fig. 40.—Goliath Overhead Traveller.

machine, constructed by Messrs. Stothert & Pitt, of Bath, illustrated in fig. 41, used for constructing a breakwater at the port of Goa in India, is of this kind. The overhang is 25 feet, measured from the front leg to the extreme position of the load; the extreme range of cross travel is 18 feet and the vertical range of lift 40 feet; the clear height under the cross girder is 16¾ feet and the working load 40 tons.*

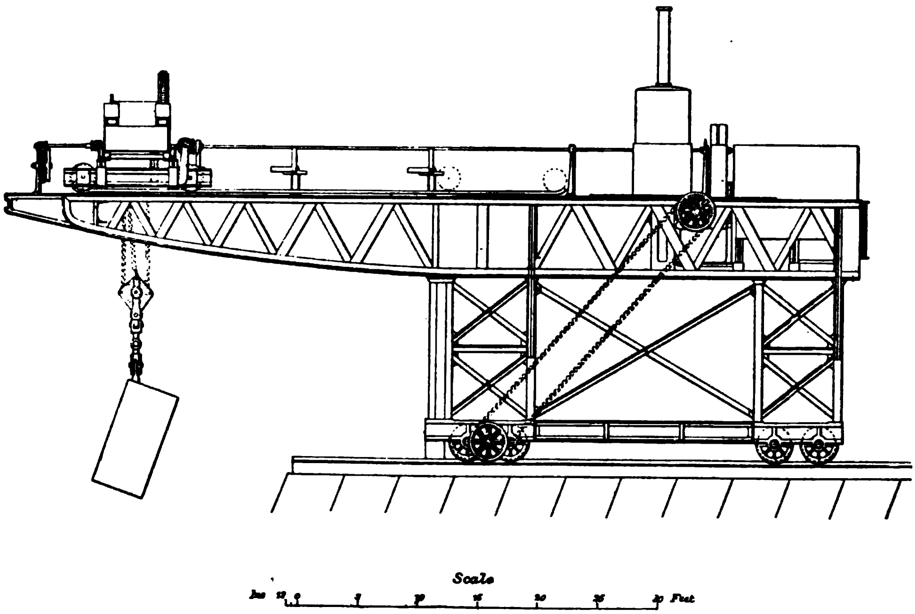


Fig. 41.—Titan Overhead Traveller.

NEGATIVE APPLIANCES.

Excavators.—The various classes of implements for the removal of earthwork, in bulk and in the dry, from the site of a proposed dock may be enumerated as

Land Dredgers.
Steam Navvies.
Grabs.

Land Dredgers are an adaptation of the principle of sea dredgers to land work. They are a foreign product, and subdivisible into two types, which may be distinguished as the French and the German respectively, according to the country of their origin. Both, however, are one in mode of action, and the distinction between them simply lies in the fact that the former discharges its load into waggons entirely to the rear, while the latter discharges into waggons which pass underneath its framing. This arrangement gives the German machine a wider base and greater stability. There

* Pitt on "Plant for Harbour and Sea Works," *Min. Proc. Inst. C.E.*, vol. cxiii.

are occasions recorded in which the French machine has overturned when working in light, marshy clays.

While the principle of the land dredger is identical with that of the sea dredger, there is a difference in the method of working. In the sea dredger the buckets excavate downwards, away from the vessel, whereas in the land dredger the cutting action is upwards, towards the machine. The buckets of a land dredger are much shallower and lighter than those of a sea dredger, but both machines are alike in that they are fitted with two tumblers, about which the buckets revolve, discharging their contents in passing over the upper tumbler.

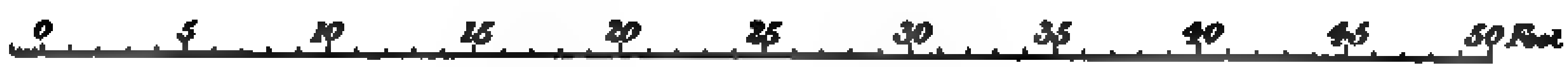
A very important advantage attaching to the use of land dredgers is the saving of a considerable amount of haulage up inclines. The machines not only excavate cuttings to a depth of 15 or 20 feet, but they also deliver the spoil at a level of 6 or 8 feet above the ground upon which they travel. This means, of course, a marked saving in time, as well as in locomotive or winding power.

On the other hand, the first cost of these machines is very great, amounting to about £2,400 each ; and they require much heavier roads than machines of lighter build. Under very favourable circumstances the cost of excavation with these machines has amounted to 1½d. per cubic yard excavated, but this figure may very easily be doubled in cases where space is circumscribed and action impeded. Such conditions often prevail in dock construction.

French Machine.—A land dredger constructed by Messrs. J. Boulet et Cie., of Paris, is illustrated in figs. 42 and 43. It was employed in excavating the site of Canada Branch Dock No. 2, Liverpool, and formed one of four engaged upon the formation of the Manchester Ship Canal. Experience showed that it is only suitable for use in connection with light soils, such as earth filling, sand, loam, and gravel. It is of no value in stiff clay or in rock, however soft. Being heavy in build (about 80 tons weight), a strong and expensive road is entailed to carry the machine upon the soft ground in which alone it is effective. For this purpose steel rails, weighing 80 lbs. per yard, are required, resting upon cross sleepers 2 feet apart, and sometimes upon longitudinal sleepers in addition.

Another important consideration is the fact that a special locomotive is required in attendance upon the machine to move the waggons along under the discharge shoot, as, although the excavator has motive power, it is not sufficiently rapid to keep pace with the rate of filling. About forty men are also required to be in attendance, tending and laying the road.

At the Canada Branch Dock the French machine has excavated 770 cubic yards of soft material in a day of ten hours, and its average has been 600 cubic yards per diem, but the area in which it worked was restricted and the material not altogether favourable, so that it did not



Figs. 42 and 43.—French Steam Excavator.

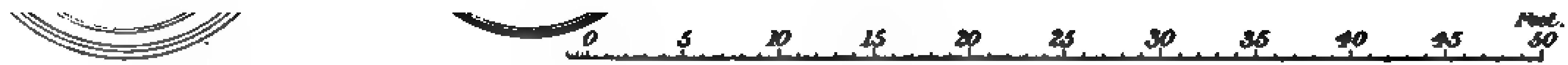
have a fair chance of displaying its maximum capabilities. On the Manchester Ship Canal, where there was much greater scope, Sir E. Leader Williams records the following as being the best single-day performances on different sections of the work:*—No. 3 section, 1,943 cubic yards; No. 5 section, 1,624 cubic yards; No. 7 section, 2,250 cubic yards; No. 8 section, 2,025 cubic yards. "These," observes Sir Edward, "are remarkable figures; but the soil and other circumstances must be suitable in order to afford such results. The average day's work on all the districts was about 1,500 cubic yards. If 440 waggons, containing 1,650 cubic yards, were filled per day on No. 8 section, it was considered a fair day's work. A bonus of a penny per cubic yard was paid to the men on everything above this quantity. For the excavation of this quantity the average daily expenses of the machine in wages of crew, coal, stores, and repairs, the last item being heavy, were about 60s., or 0·44d. per cubic yard excavated. There were employed upon the excavator an engine-driver and a stoker, and, round it, a number of men, varying from 28 to 43, the average number being 35, the roads requiring frequent moving."

German Machine.—The land dredger, illustrated in figs. 44, 45, and 46, was made by the Lübecker Maschinenbau-Gesellschaft. Similar in principle and in mode of action to the French machine, it will only be necessary to touch upon the points of difference, which are of but secondary importance. The German excavator has greater stability, owing to its broader base, and its motive power is sufficient to propel it forward at a rate commensurate with the speed of filling the waggons; hence, an attendant locomotive is unnecessary. The machine is some 10 tons less in weight than the French machine, and is generally of lighter build, but the initial cost is about the same. The following particulars of its work upon the Manchester Ship Canal are taken from the paper by Sir E. Leader Williams already referred to:—

"The best day's performances that are recorded in its favour are as follows:—No. 3 section, 2,073 cubic yards; No. 4 section, 1,736 cubic yards; No. 5 section, 1,725 cubic yards; and No. 6 section, 2,400 cubic yards. The average day's work is 1,416 cubic yards, with an average number of 36 men. The average daily expenses of the machine in wages of crew, coal, stores, and repairs are about 60s., or 0·5d. per cubic yard excavated, which is increased to 1·6d. per cubic yard by the wages of the labourers who attend on the excavator."

"Summarising the results of experience in the working of land dredgers in England, it may be said that in light material and on level ground they will fill waggons at considerable speed and with economy; and where large excavations of soft material have to be made with rapidity, the bucket dredging system gives the cheapest and best results. But they

* Williams on "Mechanical Appliances employed in the Construction of the Manchester Ship Canal," *Min. Proc. I. Mech. E.*, 1891, p. 418.



Figs. 44, 45, and 46.—German Steam Excavator.

will not excavate heavy or strong material; they are difficult and expensive to maintain, and therefore cause delay to the work; they require a costly and a heavy road, and special precautions on soft ground to prevent them from tilting over into the cutting; and they are expensive to move from one cutting to another."

Steam Navvies represent a class in which excavation is performed by a single bucket working at the end of an arm or lever. The machines travel along the bottom of the cutting, and the mode of action is an upward curved sweep of the bucket against the face of the ground in front. Steam navvies or excavators, as they are sometimes called, are characterised by great power. They are capable of working in the stiffest clay and the hardest marl. They will also take soft rock unaided, and hard rock with the assistance of a little blasting.

The *Ruston Steam Navvy*, manufactured by Messrs. Ruston, Procter & Co., Ltd., of Lincoln, has a strong spandril-shaped jib, intersected at its centre by a long arm, at the lower end of which is the bucket. The arm is capable of forward motion by means of rack and pinion gearing, and it also rotates about the pinion under the tension of a chain leading from the bucket to the head of the jib. The method of action is clearly indicated in fig. 47. The size usually employed for dock work is that developing 10 H.P., in which case the capacity of the bucket ranges from 1½ cubic yards for stiff ground to 2½ cubic yards for sand. The best results are obtained when the excavation has a depth of from 20 to 25 feet. Under such circumstances from 1,700 to 2,000 cubic yards of sand, and very dry, friable material, have been obtained in a day of 10 hours, but a fair average in mixed earth, under ordinary conditions, would be 600 to 700 cubic yards per diem. In hard material, such as rock and rocky marl, the output is necessarily less again than this. At Barry Docks from 450 to 500 cubic yards per day were excavated, the marl being first loosened by powder. Of soft material, 1,000 cubic yards were obtained in a single day, on several occasions, at the same place.

The disadvantages attaching to the machine, undoubtedly powerful and useful though it be, are its great weight (about 45 tons), which necessitates a very solid road, and its inability to work otherwise than directly forward. The waggons to be filled must be ranged alongside, as the pivot only rotates through a semicircle, and a wide base is required to accommodate two waggon roads in addition to the navvy road. The first cost of the machine is about £1,200, and the working expenses, including wages, amount to about 30 shillings per day.

The *Simpson and Porter Excavator* (fig. 48), manufactured by Messrs. J. H. Wilson & Co., Ltd., of Liverpool, is a lighter machine, but very effective in suitable soil. The special point in its favour is its ability to revolve through a complete circle, and therefore to deliver the excavated material into waggons at its extreme rear, if necessary; and further, by disconnecting the bucket gear, the machine is readily available for use

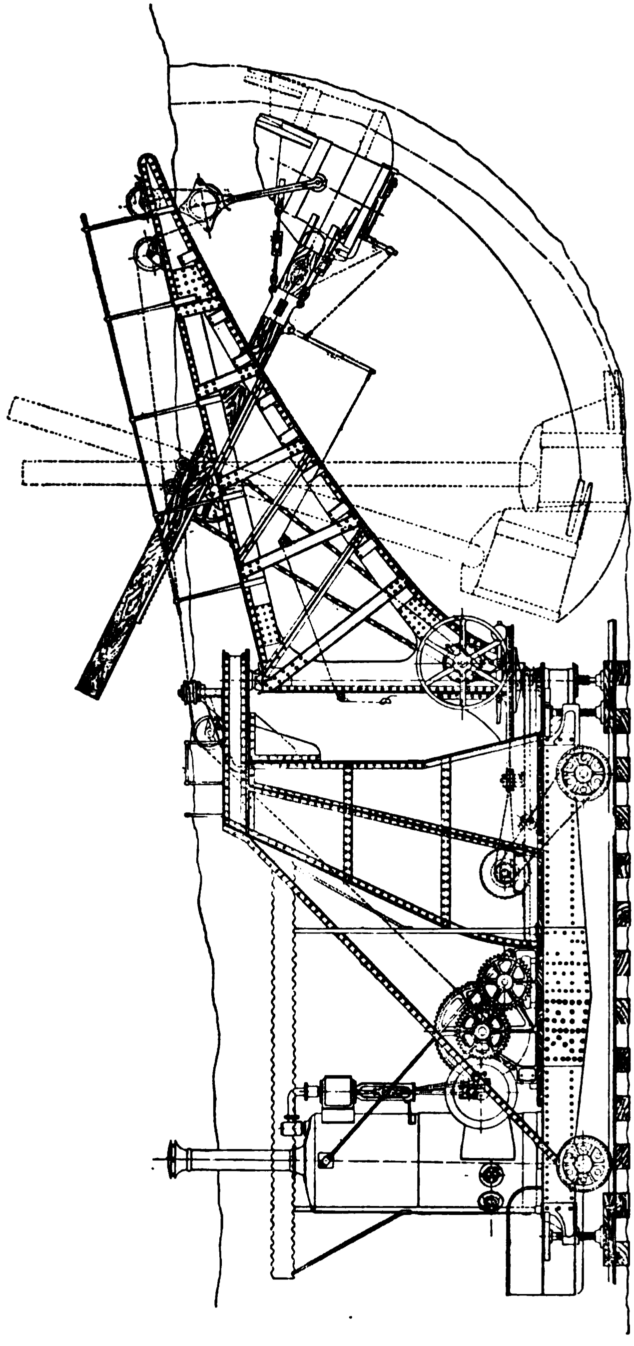


Fig. 47.—Ruston Steam Excavator.

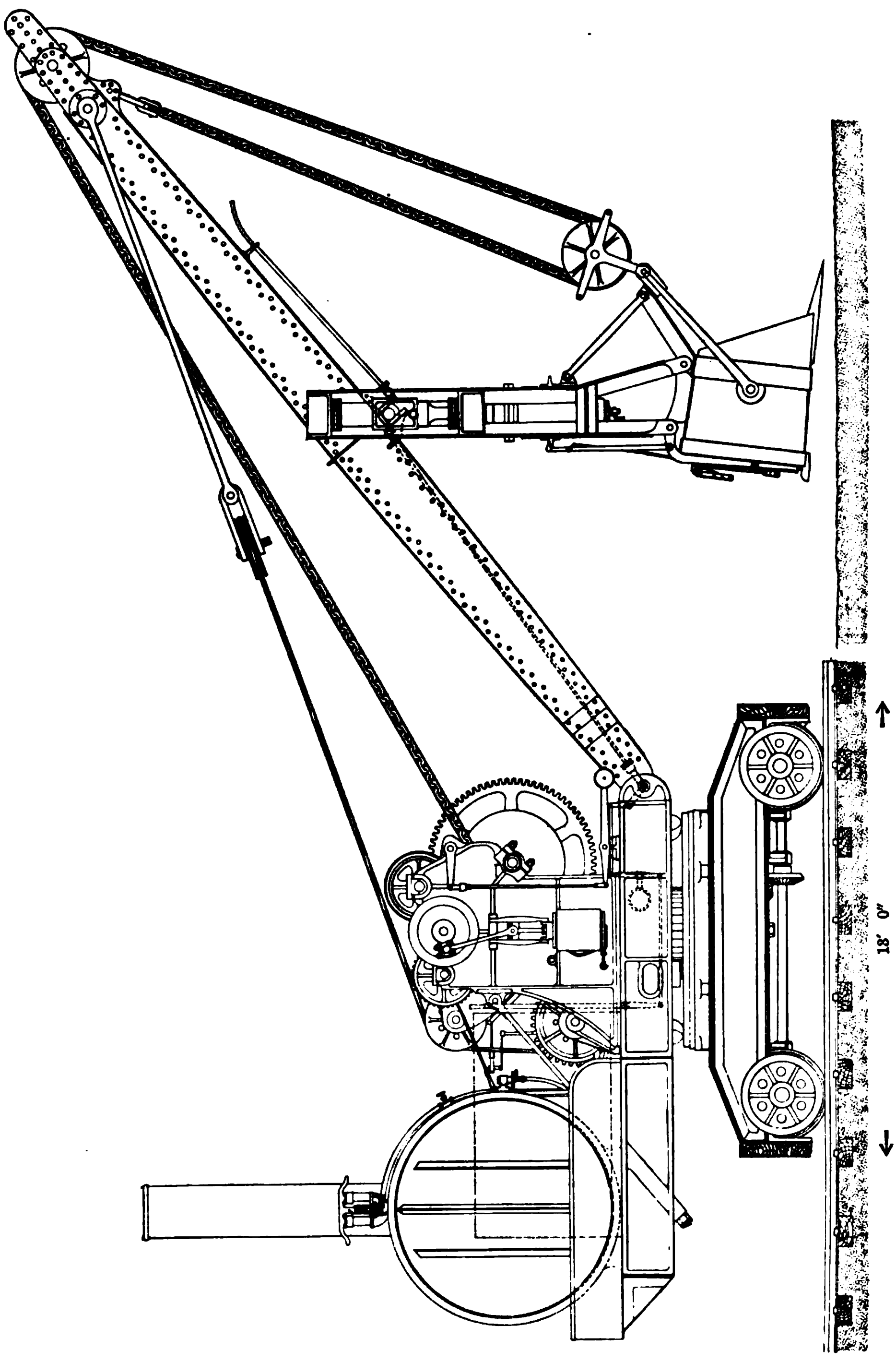


Fig. 48.—Simpson and Porter Steam Excavator.

as an ordinary crane. The rate of work claimed for a 12-ton machine, fitted with a $1\frac{1}{2}$-yard bucket, is from 800 to 1,200 cubic yards per day of 11 hours, according to the nature of the ground. It has been known by the writer to maintain an average of 570 cubic yards of stiff clay per day of 10 hours, under normal conditions, with the attendance of 10 men and 2 horses. The cost is about £1,200, and the ordinary working expenses amount to about 25 shillings per day.

The method of action is somewhat different from that of the Ruston navvy. The bucket is operated by a direct-acting steam cylinder, the piston of which has a stroke of 6 inches to 2 feet in any position of the face. Being lighter in build, the machine is not so well adapted to rock-getting as the Ruston machine.

Very similar in design is the *Whitaker Excavator*, made by Messrs. Whitaker, of Horsforth, near Leeds, and its capabilities are also about the same. It requires the attendance of a dozen men, with two horses, and its daily working expenses lie between 25 and 30 shillings. The cost of a 10-ton machine with $1\frac{1}{4}$-yard bucket is about £1,250.

A very interesting application of hydraulic power to dock construction is illustrated in the *Hydraulic Navvy* (fig. 49), designed by Sir W. G. Armstrong & Co., and used in the formation of the Alexandra Dock at Hull. The jib is similar to that of the Ruston navvy. The lifting ram and multiplying sheaves are placed, in an inclined position, at the rear of the machine, so that their weight may exercise the greatest counter-balancing effect when the bucket is making a cut. The diameter of the ram is $14\frac{1}{4}$ inches and the stroke 4 feet 5 inches. The hydraulic working pressure at Hull was 700 lbs. per square inch, which afforded a maximum cutting force, allowing for friction, of about 12 tons. The capacity of the bucket was $1\frac{1}{2}$ cubic yards, and the machine could excavate 600 cubic yards of suitable ground in $10\frac{1}{2}$ hours. Its speed of working, compared with a steam navvy, was as 13 to 10, and the ordinary repairs as 10 to 14. The cost of the machine complete was about £1,300, and its weight 30 tons. The average daily consumption of water was 17,000 gallons.*

Hydraulic appliances are not generally feasible for constructive work, unless the power be pre-existent. A contractor would scarcely deem it worth while to lay down a special installation for the purpose. But, where available, the system offers the following advantages over steam power. It is more rapid and more reliable in action, with less vibration and less noise. There are fewer repairs to be made, and, in the absence of coal and of water boilers, there is less weight to be carried over soft or uncertain ground.

Grabs are also used as excavators, but their rate of working is much inferior, and they are best adapted to confined situations and to the removal of light surface soil, under which conditions an average output of 300 cubic yards per 10-hour day has been obtained. They can excavate

* *Vide* Hurtzig on "The Alexandra Dock, Hull," *Min. Proc. Inst. C.E.*, vol. xcii.

clay, but at a much slower rate—about 100 cubic yards per diem. In accordance with their more appropriate inclusion amongst dredging appliances, a description of them is relegated to that section.

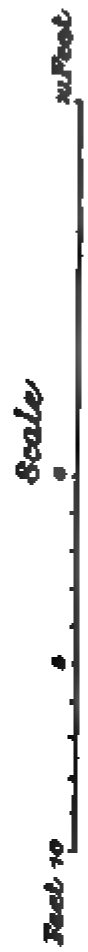


Fig. 49.—Hydraulic Excavator.

Drilling Appliances.—For the removal of rock, old masonry, and other hard material, in large quantities, blasting is the expedient commonly adopted. For this purpose, and for others, such as the insertion of the ends of jetty piles in a foundation of natural rock, &c., drilling appliances are necessary.

Drilling tools are divisible into two classes—hand drills and machine drills.

Hand drills are round bars of iron or steel, with a steel cutting edge, either cruciform or chisel shaped, and are of two sizes. The short hand drill can be manipulated by one man. He holds the drill in the left hand and strikes it with a hammer in his right. Sometimes two men are engaged—one as a holder and the other as a striker. The drill should be slowly rotated.

Long hand drills, or jumpers, necessitate the attendance of several men. If driven vertically, the drill is lifted by their combined effort and allowed to fall, being caught at its rebound and at the same time turned through a slight angle. If the cutting be horizontal, the drill is projected backwards and forwards by a swaying movement of the holders.

Hand drilled holes are from $\frac{3}{4}$ inch to 2 inches in diameter, and the depth varies, of course, according to circumstances. For blasting purposes from 2 to 4 feet will suffice. The rate of drilling depends upon the nature of the material, but may be taken between the limits of 5 to 10 feet per 10-hour day. The cutting edge will require re-sharpening, at intervals represented by from 6 to 18 inches of excavation in depth.

Machine drills are much more rapid in action than hand drills, and they also work more economically, but their installation is expensive and only justifiable in the case of extensive operations.

Machine drills are of two kinds—percussive and rotary. The former are identical in principle with hand drills, the distinction lying simply in the nature of the motive power applied, which may be steam, compressed air, or electricity. Instead of using a single cutting edge, however, several chisels may be worked in combination, especially where large holes are required. For vertical boring the drill is often surged by a wire rope leading over sheaves to a winch. The chisels vary in width up to 24 inches, but the vibration due to such a heavy chisel as this last is apt to cause frequent breakages in the rods.

Rotary drills are tubular, with extremities fitted with hardened steel teeth or diamonds, the latter being more general. The drill consists of two parts—the boring bit and the core lifter. In the course of action the former makes an annular cutting, leaving an internal core upstanding, which, when the operation is finished, is gripped by a loose toothed ring contained within, and caught in its turn by, the coned inner surface of the drill. The core, being thus jammed in the drill, is broken away at the root by a few additional revolutions.

In ordinary rock, machine drills can bore holes, 2 to 3 inches in diameter, at rates varying from 1 to 10 feet per hour.

Blasting Agents.—The agents most commonly used are:—

Gunpowder;

Nitro-glycerine and its compounds, such as dynamite; and

Gun cotton and its compounds, such as tonite.

Gunpowder is a mixture of sulphur, nitre, and charcoal. It exerts an explosive force of from 18 to 40 tons per square inch, and weighs about 62½ lbs. per cubic foot. For blasting purposes the lower power is used, and a cubic yard of quarry rock requires a charge of from ¼ lb. to 2 lbs., according to nature and position; in tunnels and shafts as much as 6 lbs. has been used.

A formula given by Haswell for computing the quantity required is—

$$\text{Charge in lbs.} = \frac{l^3}{x},$$

where l is the length of the line of least resistance in feet, and x a factor ranging from 25 for limestone to 32 for granite. The line of least resistance should not exceed one-half the depth of the hole.

Nitro-glycerine results from the action of nitric and sulphuric acids upon glycerine. The addition of a granular absorbent constitutes *dynamite*. This absorbent may be either inert or, in itself, an explosive. Dynamite, containing 75 per cent. of nitro-glycerine, has from four to six times the explosive force of gunpowder.

Gun cotton is cotton dipped in a mixture of nitric and sulphuric acids. *Tonite* is gun cotton, in a finely divided state, mixed with nitrate of barium. The power of tonite may be said to be equal to that of dynamite, but the effect is somewhat less shattering.

Haulage and Traction.—The question as to the relative merits of locomotives and stationary winding engines for the haulage of excavated material from a lower to a higher level, depends entirely upon local circumstances. Where there is ample space for the comparatively flat incline upon which locomotive traction is practicable that method is, generally speaking, preferable on the grounds of economy in working and of saving in time. The waggons can be conveyed direct from the excavator to the tipping station, whereas with the winding engine there are at least two breaks in the journey—one at the foot of the incline, where the waggons have to be connected with the hauling apparatus, either singly or in small detached groups, and the other at the summit, where they have to be disconnected and coupled up again. In the former case, under convenient circumstances, one locomotive may serve all requirements, both taking the waggons to the tipping station and bringing them back again. In the latter instance two locomotives, in addition to the winding engine, are absolutely essential—one working at the higher and the other at the lower level.

Winding engines can, however, be satisfactorily employed where space is much restricted, since the incline may practically be made at any angle and as steep as is considered desirable. A slope of about 1 in 20 represents the critical pitch at which traction by locomotives begins to lose its superior efficiency. A very steep pitch throws considerable strain upon the working parts; and, indeed, in any case, it is advisable to arrange a triangular-shaped siding in order that the engines may be reversed from time to time.

A turntable for such temporary purposes would, of course, be impracticable on grounds of expense. The waggons also need reversing at intervals, as there is a tendency for the flanges of the wheels to wear unequally when the curves of the roads have one prevailing direction. This can be done by a crane.

Waggons are of three kinds—ballast or permanent way, side-tipping, and end-tipping. Ballast waggons have fixed bodies, and thus, being steady in travelling, are employed for the conveyance of spoil to great distances. The contents, about 5 cubic yards of material each, have to be discharged by hand, unless the waggons be lifted bodily and overturned, as is sometimes done. Side-tipping waggons generally have their bodies supported on rockers formed by curved channel bars bearing upon short cross rails. They are temporarily secured by pins and catches, upon releasing which tilting becomes possible and the contents are shot out. End-tipping waggons have bodies hinged at one end to longitudinal bearers. They can be lifted in order to discharge, but are usually driven with some impetus against a wooden log fixed as a buffer upon the rails. The abrupt stoppage causes the tail-end of the waggon to jump up. The method involves, as can readily be imagined, considerable wear and tear. Tipping waggons contain rather less than ballast waggons, say, from 3½ to 4 cubic yards of material.

Dredgers and Dredging Plant.—All operations involving the removal of material under water are comprehended in the term dredging, whether the mode of action be dragging, sucking, or digging.

As a primary distinction all dredgers may be included in one of two classes:—

Compound hopper-dredgers.
Simple dredgers with attendant hopper barges.

The *hopper-dredger* is self-contained and complete in itself, being provided not only with apparatus for raising material, but also with compartments for its reception when raised. The dredger loads itself, conveys its load to the assigned position, discharges it there and returns, all under its own engine power.

An obvious disadvantage is the discontinuity of its dredging operations, with the attendant repetition of mooring manœuvres. Where new works are being carried out there is a corresponding loss of time, which is a matter of serious importance from several points of view. For maintenance works and minor undertakings the objection has possibly not so much weight; but, in either case, the drawback is emphasised by the possibility of the dredger being weatherbound and unable to leave a sheltered position in order to proceed to sea and discharge.

On the other hand, the combined hopper dredger costs less in initial expenditure and subsequent upkeep than a separate dredger and hopper with corresponding or even greater capacity. It also monopolises less valuable water space in restricted areas, such as the interior of docks. Only one crew is required to carry out all duties; the working expenses are less,

and the time taken up in sea trips may be usefully employed in overhauling the buckets and pins and in effecting any necessary repairs. A possible demur to this last contention on the ground that both machinery and crew would be too fully occupied with purely navigatory functions to admit of such extraneous duties, may be met by the explanation that repairs would be limited in each voyage to those buckets which were actually accessible, and that the presence of one or two additional hands in order to attend to them would be fully compensated for by the saving in time.

In undertakings of considerable magnitude, where time and interest on capital are factors of the highest importance, it will, on the whole, be found expedient to adopt the separate system with a large fleet of hopper barges in constant attendance upon the dredgers; for, though the outlay may be greater, the increased rapidity of execution will fully compensate for it.

Apart from the foregoing classification, dredgers are capable of inclusion in a great variety of divisions, according to the very varied manner in which they individually discharge their functions. Indeed, the subject is one of such wide scope and importance as to claim a special treatise, if anything of the nature of an adequate dissertation were to be attempted. In the limited space at our disposal we can only afford to deal in a general way with the relative merits of the more important types, and to give a brief description of their salient features. For this purpose we will adopt the following succinct classification :—

Suction dredgers.
Ladder dredgers.
Dipper dredgers.
Grab dredgers.

Suction dredgers, hydraulic dredgers, or *sand pump dredgers,* as they are very commonly called, consist essentially of a continuous pipe or tube through which, by means of suitable machinery, sand or other light material is sucked up from the bottom (see fig. 50). The sand is naturally accompanied by a very large volume of water which is delivered with it into the hopper, and this fact, combined with the disposition of the water to escape over the sides of the hopper with the sand still in suspension, causes a great deal of unremunerative pumping, the loss in sand amounting to as much as 20 per cent. of the quantity actually raised. Considerable diminution of this waste has been effected by a device introduced by Mr. A. G. Lyster, the engineer to the Mersey Docks and Harbour Board* (fig. 51). The hopper is entirely covered over with the exception of a narrow central portion, 4 feet wide, provided with adjustable coamings, raised to a height of 5 feet. The sand is delivered near the sides of the hopper, and having a considerable distance to travel before it can reach the top of the central opening, the greater portion settles *en route* and the effluent is comparatively clear. It

* Lyster on "Sand Pump Dredgers," *Min. Proc. Inst. C.E.*, vol. cxxxviii.

Fig. 60.—Suction Hopper Dredger, Seine Navigation.

should not be overlooked, however, that this arrangement, whilst extremely effective for its particular purpose, somewhat reduces the useful capacity of the hopper for solid material, by adding to the gross load carried.

The suction pump dredger would also be applicable to silt and mud, were it not that the lower specific gravity of such material renders it practically impossible to secure its deposition within the limits of the receiving hopper. Silt will take nearly as many hours to settle as sand takes minutes. It is sometimes, however, an advantage to bring a suction pump to bear on mud in situations otherwise inaccessible, such as gate platforms and recesses. The mud thus disturbed settles in more open positions, where it can conveniently be removed by other appliances. The discharge of the muddy effluent of a suction pump into a tidal or other current is a simple but efficacious means of maintaining a waterway, provided that the deposit be light and the current sufficiently powerful to retain it in suspension until it reaches a place where its settlement will do no harm.

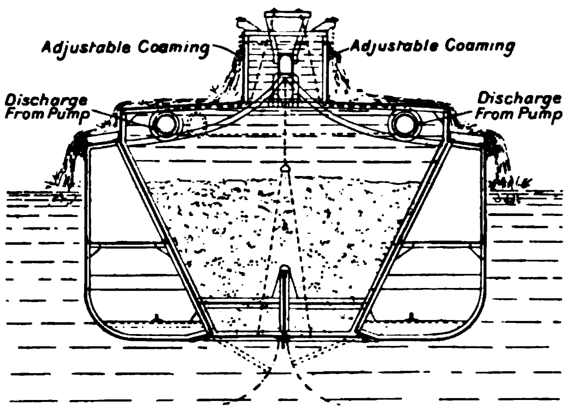


Fig. 51.—Section of Hopper fitted with Adjustable Coamings.

Suction pumps possess very great advantages in exposed situations, where the incessant motion of the waves materially interferes with the working of other forms of dredging apparatus. Equipped with telescopic pipes and flexible joints, they can adjust themselves to the rise and fall of the vessel and be quite independent of variations of level, either momentary or prolonged. The manifest convenience and safety attaching to dredgers of this class has led to repeated attempts to adapt them to the removal of material other than sand. With this object in view the lower end of the suction pipes has been fitted with a number of cutting blades, the revolution of which, by suitable gearing, is intended to disintegrate clay, marl, and other compact material to such a degree as will admit of their being drawn up the suction pipe.

This is the basis of the Bates, the von Schmidt, and other systems of dredger. The cutters, generally speaking, are cylindrical, hollow,

straight, or spiral blade milling cutters, mounted around and concentrically with the end of the suction pipe. They consist of a number of knives (from 10 to 15) united by suitable discs, or rings, at one or both ends. The whole cutter may be secured to the end of the suction pipe and rotary motion imparted to them together, or the cutter shaft may be journalled in a suitable bearing provided in the end of the suction pipe, which is then made stationary.

The use of cutters is only practicable in fairly smooth water; in situations where there is much swell, other means must be found for loosening and disintegrating the material to be removed. One alternative expedient is the application of numerous water jets through a series of orifices, specially provided for the purpose in the bars which traverse the mouth of the drag-piece, and communicating by means of suitable ports with a pipe running along the front of the mouthpiece. This system of nozzles is supplied with water under pressure through a flexible pipe. The result is much inferior to that attained by the action of cutters, and, in order to obtain the best effect, it is necessary to concentrate the pressure of the jets upon a small surface, and to direct the stream towards the intake pipe.

The value of the cutter appliance in dealing with beds of hard sand has been abundantly demonstrated on the Mississippi, the Scheldt, and the Volga. But after witnessing a number of trials of a similar type of dredger upon stiff clay, the writer is inclined to doubt the efficacy of the system in dealing with material of an argillaceous character, though he is prepared to admit that much may depend upon the precise form of cutter adopted. In this view he is confirmed by some remarks made by Mr. J. H. Apjohn at a recent engineering conference, which, indeed, are worth quoting as demonstrating the scope existing for experimental investigation.*

"The author's experience of rotary cutters has been with a dredger designed for the purpose of excavating clay for dock extension. The clay being silty, it was thought it would be easily broken up by the cutter, but this was not the case. The cutter had fourteen straight knives, set at an angle of 26° to the tangent of the circle round which they were placed and overlapping each other to a slight extent. The dredger was first operated at a small depth where the soil was brittle and the cutter proved efficient, but when the clay was reached at a greater depth, the openings between the blades of the cutter clogged with the tenacious plastic clay, with the result that the proportion of clay found in the water discharged through the pipe-line was extremely small. The cutter was then unshipped, and a width of some inches was cut off the inner edge of each blade, so that the overlap was done away with, and at the same time the circular opening at the bottom of the cutter was reduced in area. When again tried the cutter worked better, there being but little clogging between its blades,

* Apjohn on "Dredging with special reference to Rotary Cutters," Proc. Eng. Conf., London, 1903.

Figs. 52 and 53.—Bates' Dredger, with Clay-cutting Appliances.

but these did not cut the clay very well. A new cutter was then built, with narrow spiral knives, and proved to be more efficient than the first; but even with this cutter the quantity turned out per hour was never more than 60 per cent. of that contracted for. The clay, which it discharged behind the walls was in the form of nodules, varying in size between that of an egg and that of a Dutch cheese."

Notwithstanding some disappointing experiences, such as the foregoing, the clay-cutting gear has very strong partisans. Mr. A. W. Robinson * claims for a dredger, the "J. Israel Tarte," designed by himself, and working in blue clay in the channel of the river St. Lawrence below Montreal, "a world's record for output, measured by the output, of any dredger under any conditions."† And Mr. C. W. Darley, in his description of "Dredging in New South Wales,"‡ speaks of them as valuable for cutting new channels through "tough or hard clay formations." Any definite pronouncement on the value of the cutter dredger must therefore remain in abeyance, pending the completion of more extensive trials and the determination of the best form of cutting apparatus.

The illustration (figs. 52 and 53) is one of a dredger on the Bates system constructed for the Russian Government. The cutters, of which there are four, are shown at the stern. The forward end is in connection with a discharge pipe.

Ladder Dredgers, or bucket-ladder dredgers (figs. 54 to 58), consist, in principle, of an endless chain connecting a series of buckets which traverse in succession an inclined orbit, approximately elliptical, about two pivots or tumblers, excavating material at the lower tumbler and discharging it into a shoot while passing over the upper tumbler.

Bucket dredgers of this type have either one or two ladders—"ladder" being the name applied to the frame, with its roller bearings, on which the buckets travel. In single-ladder dredgers the ladder coincides with the longitudinal axis of the vessel. The ladders of double dredgers are situated at each side of the vessel.

A single-ladder dredger of the same capacity as a double dredger has the advantage of fewer moving parts and, consequently, of less working friction. The central position of the ladder also admits of a more convenient outline for the vessel, from the point of view of propulsion, and affords greater steadiness in a sea way. The broad beam of double-ladder dredgers renders it impossible for them to pass through narrow locks, though this difficulty has been overcome, in one case at least, by constructing a dredger in detachable halves.

On the other hand, a side-ladder dredger can work in greater proximity

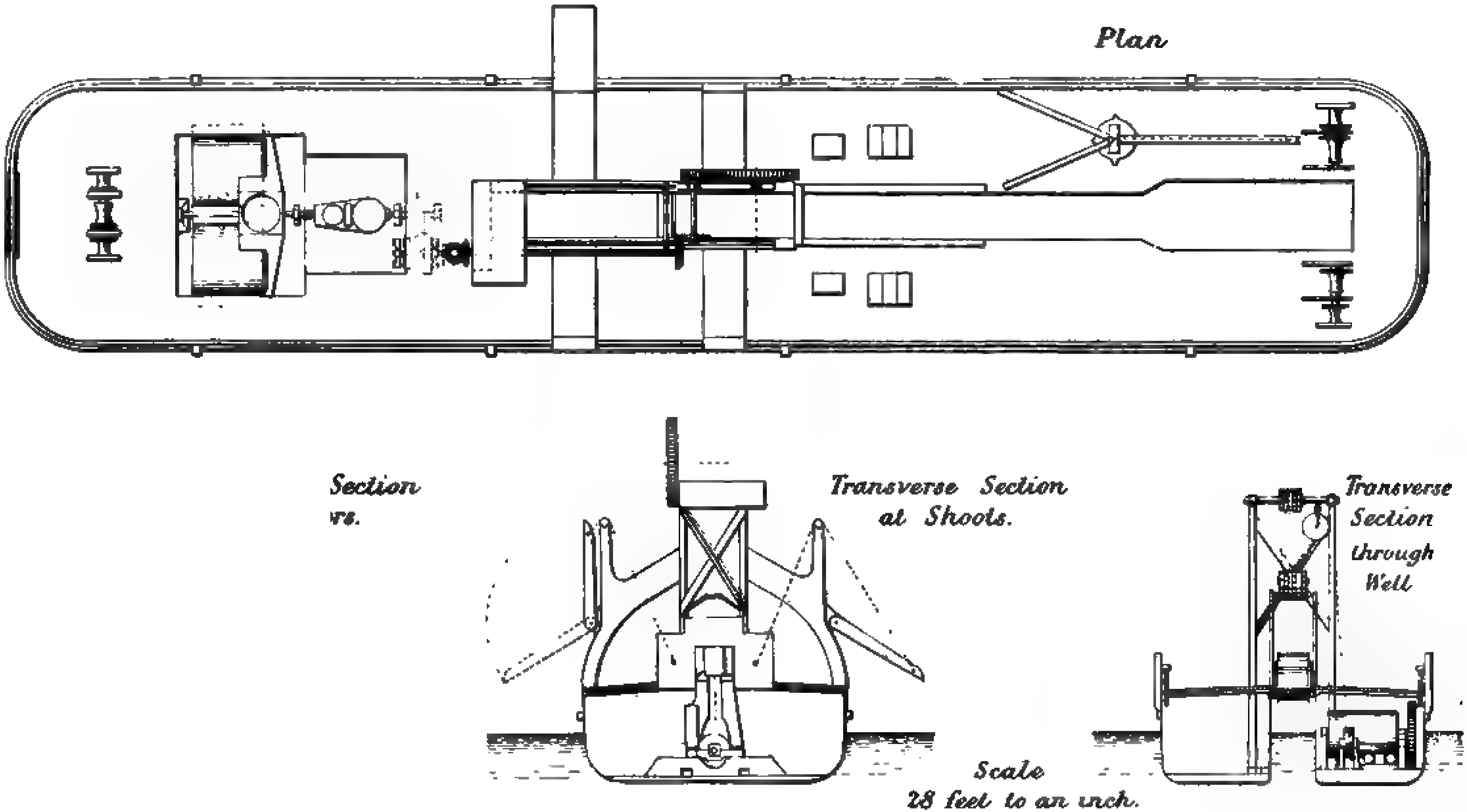
* A. W. Robinson on "Modern Machinery for Excavating and Dredging," *Engineering Magazine*, vol. xxv., No. 1, April, 1903.

† This performance is stated to have consisted in the removal of 1,180,000 cubic yards of material during a period of two months, comprising 52 working days.

‡ Eng. Conf., London, 1903.

Feet 10 5 0 10 20 30 40 50 60 70 80 90 100 Feet

Fig. 54.—Ladder Dredger "Cairndhu," Clyde Navigation.



Figs. 55, 56, 57, and 58.—Ladder Dredger "Cairndhu," Clyde Navigation.

to the face of a dock or quay wall than is feasible in the case of a central ladder. But, under these circumstances, the discharge of dredged material has to take place across the whole width of the vessel (unless it be a hopper dredger, which is unlikely, from its unsuitable form for navigation), and either the cross shoot will be too flat to be thoroughly effective, or else the lift of the buckets is excessively high for ordinary purposes. It will generally be found necessary to employ an auxiliary pump to flush the shoot.

A central ladder dredger can discharge indifferently to either side, but again, if any mishap occur to a link or bucket, the whole dredger is placed out of action, whereas in a double ladder dredger one ladder may be quite disabled without interfering with the work of the other. In cases where very powerful machines are required, double dredgers have the recommendation of providing greater lifting capacity with buckets of a less unwieldy size.

The bucket dredger is eminently suitable for steady continuous work in hard material. It is the only form of dredger which will excavate rock, and it has proved capable of raising boulders much larger than its own buckets. In stiff clay it is much superior to dredgers of any other type. Altogether, it is an excellent machine, but it cannot be worked in a swell nor in very shallow water.

It is not an economical machine in the matter of power. Owing to the necessity of discharging through a shoot, in cases where an attendant hopper is employed to receive the dredged material, lifting has to be performed by the machinery to the extent of 25 or 30 feet (the writer knows of a case of 35 feet) above the water line, representing a corresponding waste of energy.

The difficulty of dealing with shoals and banks has been solved by a special form of dredger, devised by Messrs. Wm. Simons & Co., of Renfrew, called the *traversing* bucket dredger. The ladder is supported upon a horizontal longitudinal framing, by means of which it can be projected in advance of the dredger, and thus enabled to cut the flotation of the latter through shallow places. By the same arrangement the ladder can be entirely removed from the water, and less obstruction is, in consequence, offered to its passage, when acting as a carrier hopper or otherwise.

Central ladder dredgers are themselves susceptible of subdivision into two classes, according as the well is situated at the bow or the stern of the vessel. The former is the more general type for simple dredgers, but a stern well hopper dredger derives the advantage of increased speed from a normal stem, with improved manœuvring qualities and a better shaped hull for encountering heavy seas.

The following are points of practical importance in connection with the utility of bucket dredgers.

Buckets.—No object is gained by bringing the lip of the bucket too far forward. The limit of filling will generally be the horizontal line through the inner edge when in the inclined position; hence the bucket is equally

effective with a short face as with a long one, and the former outline is better adapted for discharging. The mouthpieces, or lips, should be of hard steel rivetted to the face of the buckets which, together with the links and pins, are also of steel of special quality. A hole or two in the front is useful for the escape of water. Large buckets free themselves better than small buckets from adhesive material.

Shoots.—The least inclination for the unassisted discharge of miscellaneous material is somewhere about 1 in 4; but this is not always obtainable. With the assistance of continuous and ample flushing, together with some manual appliance, such as a pricker, the limit may be raised to 1 in 10 for mud, 1 in 15 for clay, and 1 in 20 for sand.

Tumblers.—The top tumbler actuates the rotary motion of the buckets and should be as small as possible, in order to reduce the amount of intermediate gearing. The ideal form would be the circular, but with straight links and flat backed buckets, a square or pentagonal section must be adopted. The latter is preferable, as it brings all faces of the tumbler equally in contact with the buckets. To achieve this condition with a square tumbler, an additional, or "hunting," link would have to be inserted at some point in the chain. The bottom tumbler does not transmit power and should be made of large diameter to diminish friction, say, with six or more sides. It is suspended from a cross beam on the dredger, and has to be readily adjustable to the depth of water in which the dredger may be working. For the guidance of the buckets, the lower tumbler should be provided with large flanges.

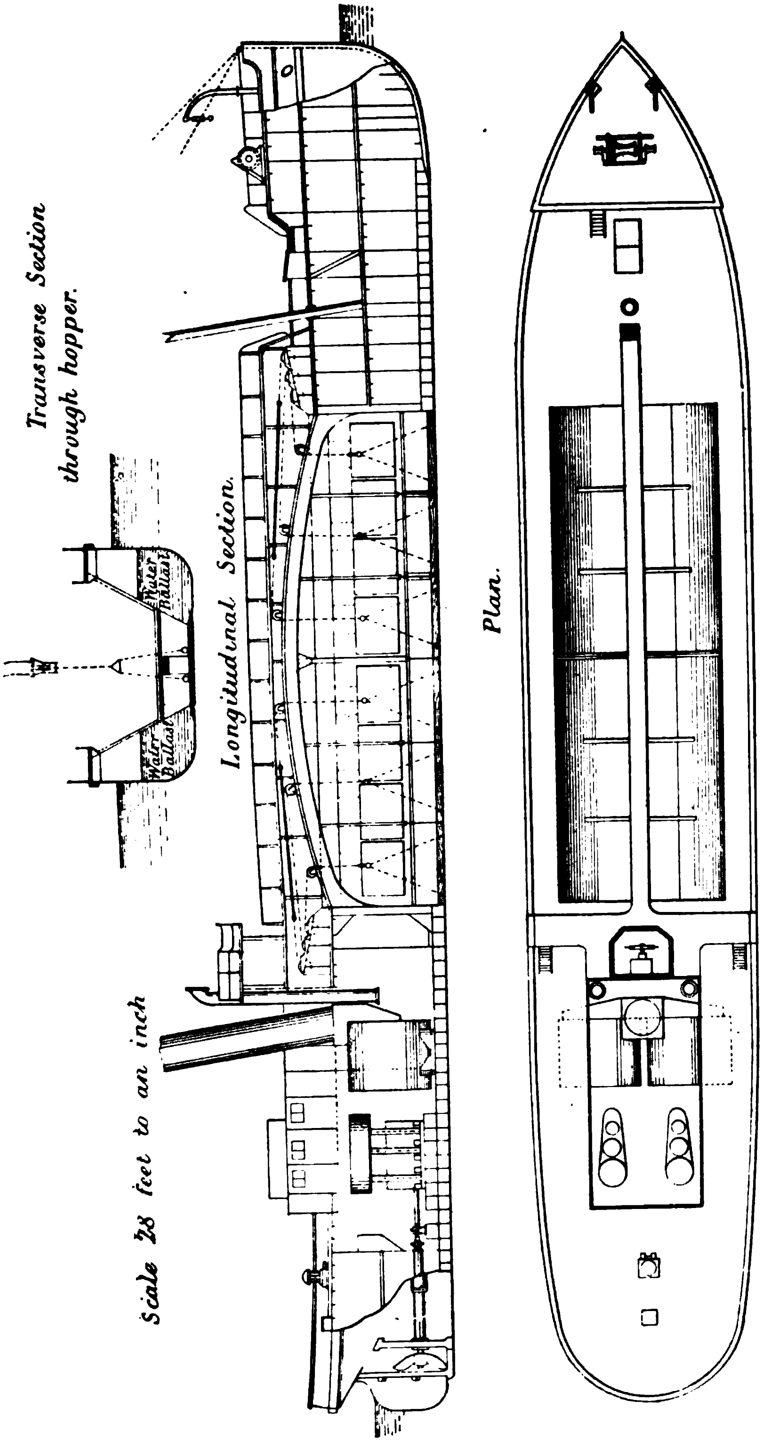
Power.—Mr. J. J. Webster,* from observation of a large number of indicator diagrams, submits the following empirical formulæ for determining the indicated horse-power required to dredge different qualities of material under varying conditions of lift. If H be the height of the upper tumbler shaft from the surface of the ground to be dredged, and W the number of tons per hour to be dredged, then the indicated horse-power required is approximately—

$\cdot 04\ W\sqrt{H}$ for very stiff clay or mud.
$\cdot 034\ W\sqrt{H}$ for hard clay and indurated mud.
$\cdot 026\ W\sqrt{H}$ for soft mud and light sand.

The illustrations (figs. 54-61) are of the dredger "Cairndhu" and one of her attendant hopper barges, belonging to the Clyde Navigation.

The **Dipper Dredger**, which is almost exclusively an American type, being much used in connection with the improvement and maintenance of river beds and channels in the United States, is so identical in principle and mode of action with the steam navvy (p. 81 *ante*), or land excavator, already described, that there is no necessity to make more than a very brief and passing reference to it.

* Webster on "Dredging Operations and Appliances," *Min. Proc. Inst. C.E.*, vol. lxxxix.



Figs. 59, 60, and 61.—Hopper Barge, Clyde Navigation.

The apparatus, consisting of a single bucket at the end of a long arm, is mounted upon a barge in any suitable position, working, for instance, either through a well in the centre, or from one end. After being lowered the bucket makes a curved upward cut, the contents being discharged into a hopper through the bottom of the bucket, which is hinged. The machine is capable of executing cuts at any required level, down to a depth of about 35 feet. Like the ladder dredger, it is not suitable for use in an exposed seaway, but it has done very effective service in sheltered positions, and when operating under favourable conditions, its capabilities may be gauged by the performances of its prototype, the steam navvy.

A machine employed in the construction of a canal connecting the rivers St. Lawrence and Grasse, with a bucket capacity of $2\frac{1}{2}$ cubic yards and excavating to a depth of 20 feet below the surface of the water, removed 138,000 cubic yards of indurated material in a period of 183 working days of 10 hours each, at an average cost of 4d. per cubic yard, including attendance, upkeep, and renewals, both for itself and the attendant barges and tug.*

Grab, or Grapple, Dredgers, known also as **Clam-shell** dredgers in the United States (the country of their origin), are essentially segmental scoops, generally two quadrants, which rotate about a central pivot, and which, on meeting in the closed position, form a semi-cylindrical receptacle or bucket. On the same principle, grabs have been constructed with spherical sides in two or three parts. This latter type is principally adapted to excavation for cylinder and circular well foundations. Either apparatus is manipulated in connection with a crane.

The grab dredger is based on two distinct systems—the single chain and the double chain. The former system is exemplified in the patents of Wild, Coles, Peters, Cooper and Holdsworth, and others; the latter in the Priestman and the Kingston dredgers.

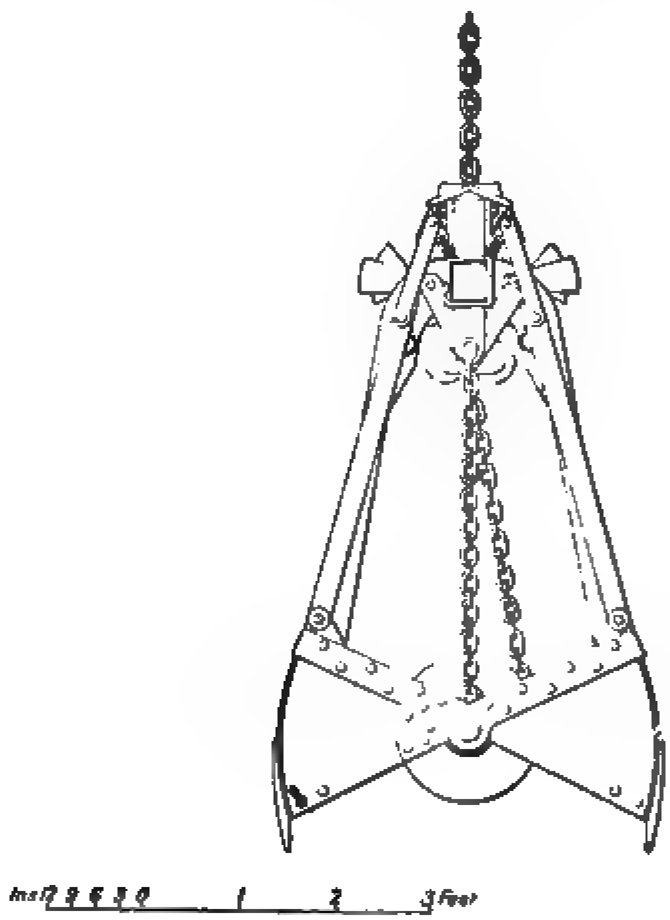
The *Wild* grab has a single chain, leading from the jib-head of the crane, fitted with a catch in the form of a half ball, or hemisphere, with the flat surface uppermost. Such a form permits the downward passage of the catch between two small tumblers, but prevents its rising again, and the grab from closing, until the bottom is reached, when the chain becomes slack and the tumblers are opened by the weight of a sliding sleeve. The grab can then be closed and drawn up until it reaches a point where a ring in the lifting gear engages two steel hooks, from which the grab is suspended whilst being discharged. The hooks are withdrawn by a simple contrivance when the grab is slightly lifted.

The action of the *Peters* machine (figs. 62 and 63) depends upon the gripping of the lifting chain, prior to the opening process, by a pair of steel arms, which are actuated and controlled by a roller, bearing against the chain, and a governing rod, attached to the upper edge of the bucket.

* Bogart on "Dredging Machines in Recent Excavations in Large Magnitude," Ninth Int. Navigation Cong., Dusseldorf, 1902.

In excavating, the bucket is closed by the chain, which continues taut during lifting. When the chain is slackened the roller falls and allows the grippers to engage. Then, on hoisting, the grab is pulled open.

In the double chain system of the *Priestman* type (figs. 64 and 65) the outer corners of the bucket are connected, by hinged arms, to a horizontal bar, or cross piece, which is capable of vertical movement in the central groove of the frame. One chain from the jib-head is attached to this bar, and any tension in it causes the bucket to open; the other chain, from the jib-head, is wound round a drum on the pivot, the unwinding of which, with the assistance of two subsidiary chains connected to the horizontal bar previously mentioned, pulls the latter down and causes the bucket to close.



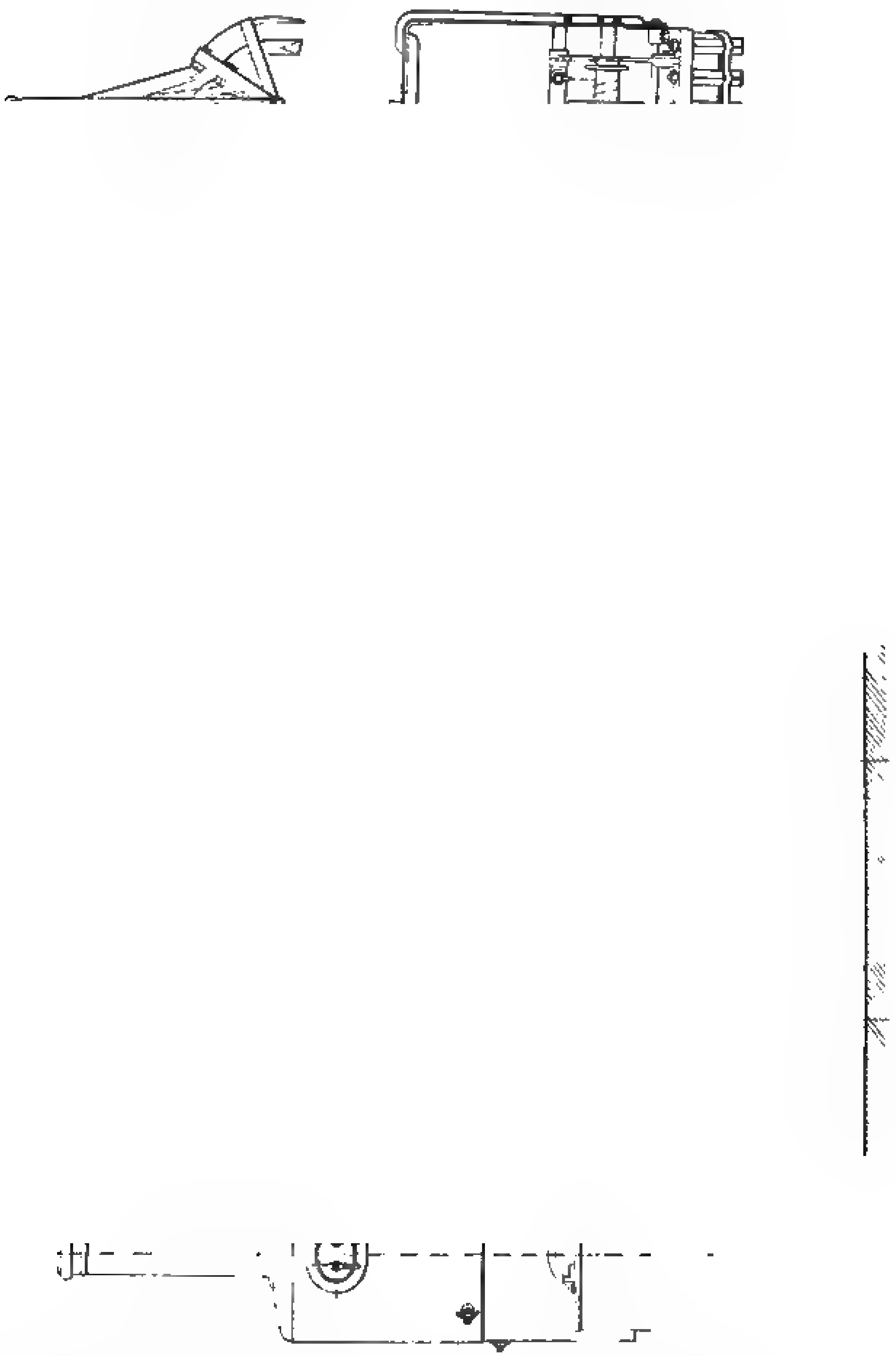
Figs. 62 and 63.—Section and Elevation of Peters' Grab.

The single chain system has the advantage of being affixable to an ordinary crane, while the double chain system involves the provision of a special crane, but it has the following important points in its favour:—

1. It has fewer working parts, and those of less delicate adjustment.

2. The opening and closing of the bucket can be effected at any point in the lift, whereas, with the one exception of the Peters machine, a single chain grab has always to be lifted to the height of the suspending piece before discharge can be made. If the latter should close upon some immovable object below water, it could not be opened again without lowering the suspending piece, or without the aid of a diver. A false or empty lift has to be completed as well as a full one.

3. The strain upon a single chain from some unseen obstacle might cause a fracture, with the consequent loss of the bucket. With a double chain



Figs. 64 and 65.—Priestman Grab.

the risk of fracture is diminished, and loss of the bucket could only occur in the event of both chains giving way.

4. A double chain grab can discharge its load as gradually as may be

considered desirable, whereas the action of a single chain grab is instantaneous.

A grab dredger with a bucket capacity of 1 ton may be assumed capable, from actual trial, of raising from 50 to 60 tons of mud per hour, when working in from 15 to 20 feet of water. Of clay, very little more than one-half this amount can be reckoned upon.

The grab is an excellent tool and invaluable in confined situations, but it is scarcely suitable for general adoption in works on a large scale. It is not an economical instrument for the removal of stiff clay; its best performances are in regard to mud and soft earth. It cannot be counted upon to work with the same regularity and evenness as the ladder dredger; in fact, its tendency is to pit the surface of the ground with a series of hollows and depressions. But, in spite of these drawbacks, it has demonstrated its utility to such an extent that it is looked upon as an essential accompaniment of most dock and harbour undertakings.

Cost of Dredging.—The conditions prevailing in regard to dredging are of so variable a nature that no information respecting the cost, at any locality, is of much use elsewhere. Further than this, the available statistics are far from uniform, and there is considerable diversity of extent in the operations included. It can only be said broadly that, within ordinary limits, dredging is effected at some price between a penny and half a crown a cubic yard, distributed, roughly, somewhat as follows:—

Suction dredging, 1d. to 6d.
Grab dredging, 3d. to 8d.
Bucket dredging, 6d. to 2s. 6d.

These figures do not apply to rock-dredging, the cost of which exceeds the higher limit, often very considerably.

AUXILIARY APPLIANCES.

Dams.—In dock construction, a dam is a temporary contrivance for the exclusion of water from a site during the progress of the undertaking. It is accordingly composed of material susceptible of easy removal, either in bulk or in parts. Timber and clay form two of the most prominent substances for the purpose. Stone and concrete are occasionally used, under restrictions to be noted later. Iron is rarely employed, and then only with a view to its ultimate incorporation in the permanent work.

In spite of its temporary character, a dam should be substantially made. The damage and delay, to say nothing of possible loss of life, resulting from the failure of any part of it, during a critical period, would far more than counterbalance any economy in construction. Too much stress cannot be laid upon this point. It is infinitely better to err on the side of excessive strength than to run the risk of disaster through an insufficient margin of stability.

In nearly every case, clay is the material mainly relied upon for the staunchness of a dam. It must be judiciously selected: free from stones, roots, and soil; not of a marly or brittle nature, but tenacious and adhesive; well tempered, watered, and worked to a proper consistency. When these points are carefully attended to, the resultant clay *puddle*, as it is termed, is capable of forming a thoroughly impervious barrier. If clay of an inferior quality be used, there is sure to be trouble with leaks and inbursts.

Temporary dams may be classified according to their composition, as follows:—

Earth dams.
Timber dams.
Stone dams.
Concrete dams.
Iron dams.

Earth Dams are peculiarly appropriate to situations where there is ample space and where a very slight elevation is required, as in shallow water. They simply consist of a mound of clay, or of a hearting of earth, covered with an outer layer of clay, deposited by tipping from waggons, skips, or hopper barges. Under the action of tipping, the mound has a decided tendency to subside, and this is still further accentuated by the softening effect of water upon the material, so that, in any case, long flat slopes are inevitable, and hence plenty of room is an absolute necessity for this class of dam. It is advisable where the natural surface of the ground is mud or silt, to excavate the site of the dam down to a solid stratum, better able to support an imposed load and to make a watertight joint with it. This last is an important point, as, if the stratum below a dam be pervious, water may be forced through it under external hydrostatic pressure. An example of an earth dam is given in fig. 186.

Timber Dams are frames of woodwork with or without an enclosure of clay puddle. They are subdivisible into

(*a*) Skin dams or sheeting dams.
(*b*) Cofferdams.

Skin Dams consist of a single row of sheeting piles, of whole or half timber, retained by tiers of horizontal walings. Lacking sufficient stiffness in themselves, they have to be supported by perpendicular or raking shores abutting upon a firm surface. Skin dams are very suitable for adoption in front of quay walls which it is desired to underpin, reface, or repair. In such cases the wall forms a convenient surface for the shore abutments, and the outer hydrostatic pressure is transmitted to the wall through the medium of the shores. The walings should be spaced at intervals corresponding as nearly as possible with the extent of zones of equal hydrostatic pressure. The amount and distribution of this pressure is calculable upon the same principles as those formulated in Chapter viii., for dock gates.

On grounds of stiffness and strength, whole timber piling is preferable to half timber piling, though a method very commonly adopted is that of driving whole timber guide piles, with intervening bays, or panels, of half timber piles. The guide, or king piles are provided with pointed shoes, but the intermediate piles are shod with wedge-shaped shoes. If an edge or side of each pile foot be splayed, the process of driving will cause it to draw more closely to the adjoining one, and so produce continuous contact. For the same reason it is a good plan to pitch or set a whole bay of piles and slightly drive them all, before proceeding to a conclusion of the process with any one of them. Furthermore, the sides of adjoining piles may be alternately tongued and grooved or, alternatively, both grooved, for the reception of a vertical strip of flat iron, say, from 2 to 3 inches wide by $\frac{1}{2}$ inch in thickness. The former method is of greater service for maintaining the regularity of the piles in driving.

Skin dams need not necessarily be piled. A method very successfully practised at Liverpool (fig. 160) is that of constructing skin dams ashore, in flitches of 100 lineal feet or more. They are then launched from the quay, up-ended with the aid of a floating crane and some iron rail ballast, and inserted in a trench previously dredged to receive them. The dam is finally shored to the wall at uniform intervals, forming bays of from 10 to 12 feet in length. The edges of adjoining piles are rendered a watertight joint by means of 1-inch triangular wooden fillets nailed to the piles and closely cramped together. Torch-wick has also been used as a watertight packing. These flitches proved very successful and were used repeatedly, being transferred from one site to another as occasion required. A length of over 4,500 feet of dock walls was underpinned in this manner. The cost of the flitches, including maintenance and removal, varied between £13 and £18 per lineal foot.

A skin dam has been made self-supporting by constructing it in the form of a bottomless box for work which could be carried on in the interior. The outer faces then afford one another mutual support through the medium of cross shores and struts. The method as applied to the construction of a dock wall at Liverpool is shown in fig. 133. It will be noticed that the outer sheeting consists of a series of horizontal timbers, ranging in thickness from 12 inches at the bottom to 3 inches at the top. Water-tightness is effected by means of torch-wick joints. Inside the sheeting there is a continuous row of piles driven down to a rock substratum, and acting as a support for an overhead crane road. The dam in question was 246 feet long, in 15-foot bays. The cost was rather less than £35 per foot run.

In all cases the foot of a skin dam has to be amply protected and covered by a thick layer of clay puddle, which will need replenishing from time to time as the clay subsides.

Cofferdams consist essentially of two timber faces enclosing a hearting, generally of clay (fig. 66), but occasionally of stone. They are of more solid construction than skin dams, but, at the same time, they offer some risks of

failure from which the former are exempt. The continual subsidence of the clay hearting involves more than the mere replacement of the disappearing material, since the latter in sinking exerts powerful pressure of a hydrostatic character against the sides of the dam, producing a strong tendency to rupture, which has indeed taken place in at least one instance to the author's knowledge. Again, the presence of horizontal walings in the interior of the dam for the guidance of the piles in their descent, and of transverse ties, is a source of much troublesome attention, because the clay, in settling, leaves cavities and interstices immediately underneath these parts, which serve as channels and ducts for leakages. The evil may be minimised by the withdrawal of the inner waling, after the driving of the piles and before the insertion of the clay, also by the substitution of timber diaphragms, extending from top to bottom, for transverse tie-rods. Where rods or bars are used, several flat washers or plates of large area with perforations near the upper edge, for the insertion of the through bolts, will sometimes serve to check the passage of water in case of a slight sinkage of the clay.

From these considerations it is clear that no useful object *per se* is served by any great thickness of clay puddle; the disruptive force is only increased thereby, and sources of leakage are more difficult to locate. A minimum width of 5 feet in the interior of a cofferdam will generally prove an adequate allowance for impermeability, but, on the other hand, as regards the stability under external pressure, the height of the dam will exercise most influence in determining its width, though this factor can be discounted to some extent by the use of auxiliary shoring.

The only external force at work upon a cofferdam is the hydrostatic pressure against its outer face. If we call this P, the height of the water h, and the weight of a cubic foot w, then the pressure per foot run (as explained in detail in Chapter viii.), is

$$P = \frac{wh^2}{2},$$

and the overturning moment about the base, the centre of pressure being at one-third of the height from the ground,

$$M_0 = \frac{wh^3}{6}.$$

Now, the dam derives its stability in varying proportions from three sources. These are—(1) its dead weight or inertia, treated as a heavy, detached mass; (2) its resistance to transverse stress, treated as a cantilever firmly fixed in the ground; and (3) the support afforded by the external strutting, if any.

(1) The moment of resistance due to the intrinsic weight of the structure is

$$M_1 = w \times \frac{b}{2}, \quad . \quad . \quad . \quad . \quad . \quad (6)$$

where w is the weight of the whole dam per lineal foot and b the breadth of the base.

(2) Considered as a loaded cantilever, the outer row of piles will be subjected to tension and the inner row to compression, or both rows will be subjected to tension and compression alike, according to whether we treat the structure as rigid or deformable. Assuming the former condition, if a be the sectional area of single piling per foot run and f_t and f_c the resistance of the material per unit area to tension and compression respectively, then the linear moment of resistance is

$$M_2 = af_t b \text{ or } af_c b. \quad . \quad . \quad . \quad . \qquad (7)$$

Assuming the latter condition, the resistance of each row of piles must be considered disconnectedly, and

$$M_2 = \tfrac{1}{6}\, af_t d \text{ and } \tfrac{1}{6}\, af_c d, \quad . \quad . \quad . \qquad (8)$$

where d stands for the depth, or thickness, of the piles.

Fig. 66.—Cofferdam at Liverpool.

(3) If α be the sectional area of one of the external struts at a distance, δ, from the base, and s the horizontal distance apart of the struts, then the linear moment of resistance due to any number of such struts is

$$M_3 = \Sigma \frac{\alpha f_c \delta}{s}. \quad . \quad . \quad . \quad . \quad . \quad . \qquad (9)$$

This is on the assumption that the struts lie directly in the axis of stress. Should this not be the case, and the angle of inclination to the horizontal be θ, we must write

$$M_3 = \Sigma \frac{\alpha f_c \delta \cos \theta}{s}. \quad . \quad . \quad . \quad . \quad . \qquad (10)$$

A similar and additional modification would have to be made if the struts were also raking on plan.

Combining all these elements, we have for equilibrium

$$M_0 = M_1 + M_2 + M_3.$$

The exact distribution of stress being indeterminate, a very large factor of safety is essential.

The stress in the internal tie-rods can only be adequately covered by assuming the clay to be in a fluid condition and exerting a pressure proportionate to its specific gravity.

Fig. 66 shows a cofferdam as employed in dock construction at Liverpool. It was straight in plan between its extreme abutments for a total length of 260 feet, divided into 15-feet bays by cross diaphragms of 3-inch planking, thus obviating the use of internal tie-rods. The height was 38 feet and the bottom and top internal widths 20 feet and 12 feet respectively. It derived some additional support from raking shores not shown in the figure. A dam of this type can be constructed, maintained, and removed at a cost ranging from £35 to £50 per foot run, much depending upon the nature of the site and the duration of the work.

Fig. 67.—Cofferdam at Hull.

Fig. 67 shows a cofferdam used at the Alexandra Dock, Hull. It was segmental in form, with a radius of 255½ feet and a length of 461 feet. The piles were driven vertically, enclosing a space 5 feet wide. Five sluice openings were formed to allow the tide to flow in and out until the completion of the dam.*

* Hurtzig on "The Alexandra Dock, Hull," *Min. Proc. Inst. C.E.*, vol. xcii.

Fig. 68 is the section of a cofferdam adopted at Limerick in connection with the rebuilding of a dock wall which had failed, the length being 450 feet.*

Stone Dams are similar in construction to earthwork dams, consisting of a mound of light stone rubble (such as chalk) deposited and overlaid with clay to form a watertight skin. This material is also used as filling for the interior of a cofferdam, as exemplified at Ardrossan harbour † (figs. 69 and 70).

Concrete Dams.—A somewhat novel and ingenious experiment in dam construction has been successfully tried at Liverpool. A wall was built of large concrete blocks (each containing 100 cubic feet) bedded in hydraulic mortar, with a sheet of ordinary brown paper laid between the blocks in each joint. The paper adapted itself to the surface of the bed and allowed the blocks to obtain a uniform bearing upon one another, while at the same

Fig. 68.—Cofferdam at Limerick.

time it prevented any actual adhesion. The stability of the structure depended, therefore, entirely upon the resistance of the blocks to sliding friction, which proved to be ample for the purpose. The dam in question was built upon the outer sill of a lock, 100 feet wide between side walls. The sill had a straight outer face and a curved inner one for the ultimate reception of gates. The area of the sill was nearly 400 square yards, with a minimum width of 25 feet. The front of the dam was a vertical plane, the back being stepped. The total height above the sill was 42 feet, at which level a roadway was formed for traffic. High water of ordinary spring tides

* Hall on "Limerick Dock Walls," *Min. Proc. Inst. C.E.*, vol. ciii.

† Robertson on "Ardrossan Harbour Extensions," *Min. Proc. Inst. C.E.*, vol. cxx.

came up to 33 feet above the sill, but during equinoctial gales the waves frequently surged to the top of the dam and broke over the roadway.

Iron Dams usually take the form of caissons, but they are by no means common. The most striking instance of their adoption is perhaps in connection with the construction of the Thames Embankment. The caissons were of wrought-iron in half oval segments, with upright flanges at each end, so that when the halves were bolted together they formed a complete oval, 12 feet 6 inches long by 7 feet wide in the centre and 4 feet 6 inches deep. The plates were $\frac{1}{2}$ and $\frac{3}{4}$ inch thick. Angle irons were bolted round

Fig. 69.—Dam at Ardrossan.

Fig. 70.—Dam at Ardrossan.

the top of the rings, enabling them to be firmly secured to each other in the vertical position. A watertight joint was formed by a guide pile, $10\frac{1}{2}$ by $6\frac{1}{4}$ inches section, fitting into a groove between adjoining caissons. The dam was further stayed by a few surrounding piles which maintained the caissons rigid and vertical in their descent. The gross cost of this dam was £30 per lineal foot as compared with £20, the gross cost of a timber cofferdam in a similar position. Some of the iron caissons were incorporated in the permanent work at an allowance of £8 per lineal foot. With this qualifi-

cation it may be added that the nett costs of the two dams were about £15 and £17 respectively.

For tidal work a dam may be conveniently contrived by sinking iron pontoons and banking them up and between with clay. The height of such a dam is necessarily small, but it materially increases the period of working within the enclosed area.

Pumps.—The subject of pumping demands the most careful and earnest attention of the dock engineer, seeing that the practicability and success of his undertakings depend largely upon the efficiency of his pumping arrangements. Some evidence of this will be afforded in subsequent chapters, but the fact is almost sufficiently obvious in itself.

There are many varieties of pumps on the market, each with its own special features and capabilities. A study of the catalogues of well-known manufacturers will generally enable a satisfactory selection to be made for the particular purpose required, and the following remarks are simply appended by way of indicating such practical points as seem worthy of consideration in exercising a choice.

Valve Pumps—that is to say, lift pumps and force pumps, or any combination of these in which the action depends upon the alternate opening and closing of small valves—are only suitable for comparatively clear water. Water which is highly charged with solid matter in suspension and with floating objects is very likely to derange these delicately adjusted parts and to put the pump out of action. The gritty nature of sand causes excessive wear of the leather washers and packings, necessitating frequent renewals. Chips and gravel lodge in the valves and prevent them from closing. The jambing of the bucket packings may cause serious trouble owing to the great force frequently required to release the bucket. For drainage purposes in trench excavations, a lift pump has this advantage over a force pump, in that, if the working should by any accident become suddenly flooded, the lift pump can still discharge its function, being actuated from the summit level, whereas the machinery of a force pump is in the bottom and, consequently, would be submerged.

A very handy drainage pump for use in confined situations is the *Pulsometer*. It represents a rather unusual principle in pumping. The action consists in the alternate admission and exclusion of steam to and from adjoining chambers. The water is forced out of one of the two chambers by steady pressure until it sinks to the level of the discharge orifice, at which point the steam obtains a free vent, and being in contact with a large surface is so rapidly condensed as to cause a vacuum in the chamber and draw over the steam ball at the top which closes the aperture and transfers the supply to the next compartment. Meanwhile, continued condensation in the empty chamber increases the vacuum, which is filled by a fresh supply of drainage water through the lower valve leading from the suction pipe. The apparatus is compact and easily suspended by a rope or chain in any desired position.

Other appliances for dealing with small quantities of water are the simple *hand-pump* and the *ejector*. The former is of the ordinary bucket type of pump, worked by hand. The ejector is actuated by hydraulic or by steam pressure. The principle is that of forcing a small jet or current

Fig. 70*a*.—Pulsometer.

A, Pump chamber.
B, Air chamber.
C, Suction pipe.
D, Discharge orifice.
E, Inlet valve.
F, Outlet valve.
G, Stop for valve.
H, Cover.
I, Steam inlet valve.
J, Neck.
K, Steampipe.

through a nozzle in the interior of a discharge pipe of slightly greater diameter. Drainage water is drawn up from the sump, by suction, to fill the vacuum thus created.

For removing "slurry" or liquid mud, water charged with sand, gravel, cement scum, floating material, and, in fact, the general drift and debris

which find their way into a pumping well in excavations carried on under circumstances, perhaps more peculiarly characteristic of dock work than of any other branch of engineering, pumps of the strongest and simplest construction are advisable. Such, for instance, are the *chain pump* and the *centrifugal pump*.

The first of these which has demonstrated its utility from remote ages, being originally an invention of the Chinese, consists of a series of flat blades, strung at regular intervals upon two parallel endless chains. These chains hang vertically, being suspended from a revolving reel or barrel at the summit, over which they travel continuously. The descent is in the open, but on reaching the bottom the blades enter the splayed orifice of a rectangular funnel extending upwards to the point of discharge. The blades fit the interior of the funnel sufficiently closely to take the bulk of the enclosed water with them without incurring excessive friction against the sides. The pump acts admirably in lifting with absolute impartiality water, mud, pieces of brick, wood, stone, and concrete; any substance, in short, which can enter the funnel. The only thing to check its action is the intrusion of a chance wedge or plank end, transversely, between the buckets and the orifice The blades, which are of wood, are, of course, subject to a considerable amount of abrasion and have to be replaced from time to time, but repairs of this kind are easily effected. A stock of fresh blades is kept at hand, and the operation of removing a damaged blade is simply that of taking out the split keys which hold it in position on the chain.

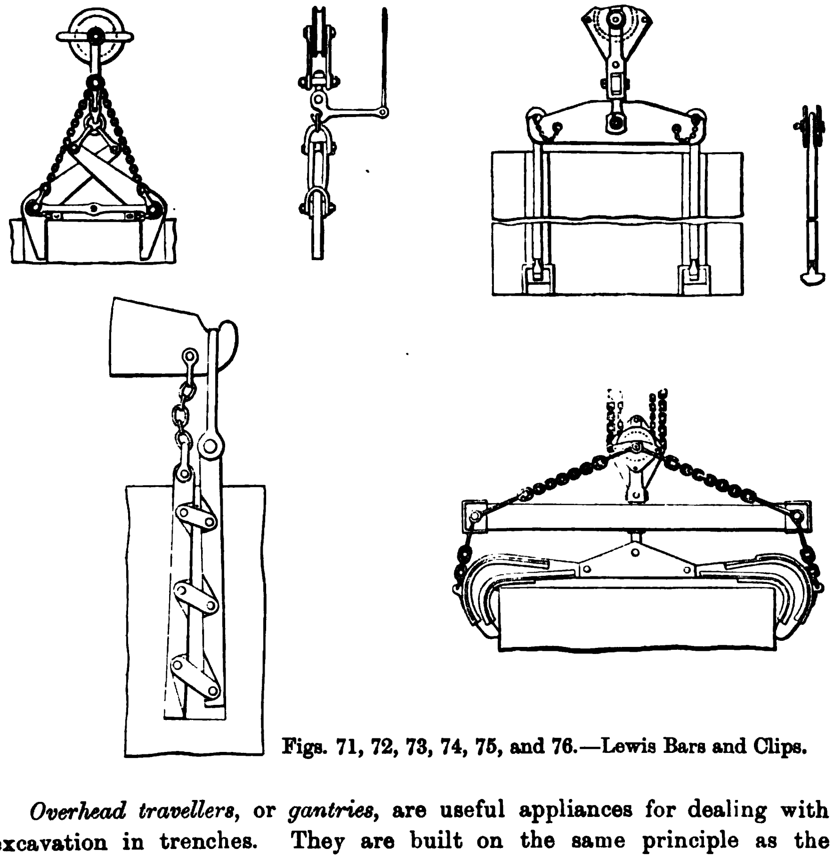
Chain pumps with rectangular blades, 2 feet long and 6 inches wide, 14½-inch centres, running at a speed of 500 feet per minute have proved capable of discharging regularly 600 tons of water per hour, which represents an efficiency of slightly less than 70 per cent. The speed may be increased to 600 or 700 feet per minute, with a corresponding greater discharge, but such speeds throw an undue strain upon the apparatus.

The action of a *centrifugal pump* is the revolution of a series of blades radiating from a common axis, by means of which the water is whirled round in a confined space until it acquires sufficient velocity to be projected up the discharge pipe. The blades are short, thick, and curved in form. This class of pump will "throw" a good deal of extraneous material, but there is always the possibility of a fairly large object being drawn through the suction pipe and getting jambed in the blades, which are less accessible for repairs than those in a chain pump. The usual sizes of such pumps for temporary duties varies between 6 and 18 inches diameter.

Before leaving the subject, it will be well to observe that the provision of a duplicate pumping system is a commendable arrangement. One set of pumps might easily break down at a critical moment, and even if the amount of pumping is sufficiently small to allow adequate intervals for cleaning and repairs, yet an auxiliary pump is an advisable precaution for unforeseen contingencies.

The placing of pumps upon the framework of dams, though sometimes unavoidable, is always to be deprecated. The vibration set up by the machinery inevitably causes settlement and induces leakage.

Cranes for constructive work are mostly of the locomotive type, and the power usually ranges from 3 to 10 tons lifting weight. The heavier machines are fitted with two gearings, by which a light load can be lifted speedily, or the full working load at a more moderate rate. There are four motions—travelling, jibbing, lifting, and slewing. For raising and depositing heavy loads within a short distance, derrick cranes may be employed. Owing to the broader base afforded by their outlying arms, these cranes have greater stability than the locomotive cranes, but they lack the rapid travelling movement of the latter.



Figs. 71, 72, 73, 74, 75, and 76.—Lewis Bars and Clips.

Overhead travellers, or *gantries*, are useful appliances for dealing with excavation in trenches. They are built on the same principle as the Goliath, illustrated in fig. 40, but are generally much lighter, and the lifting power, in the generality of cases, does not exceed about 15 tons.

Skips are buckets of various forms, used for the transfer of material by means of cranes or travellers. They hold from ½ to 1¼ cubic yards, and

are generally either round with a pivoted handle, or square with a hinged bottom.

Lewis bars and *clips*, for the lifting of masonry and concrete blocks, are of various designs. In the former case, the hold is obtained either by turning the bar through an angle or by wedging it. In the latter case, the tension in the chain causes a closing of the jaws, and the block cannot be released until the chain is slackened. A number of them are illustrated in figs. 71-76.

Constructive Plant at Keyham Dock Works.

As an example of the variety and amount of plant required for dealing with dock work on a large scale, a statement of the plant used at Keyham Dockyard Extension Works is quoted from Mr. Whately Eliot's paper on the subject:—*

"The works occupy ground to the extent of 113 acres, of which 35 acres are situated above high water mark, being chiefly land which has been reclaimed, in former years, from that part of the Tamar called the Hamoaze. The remainder of the area, 78 acres in extent, is the foreshore of mud from high water line to about low water of spring tides, the range of tide being 15½ feet. The works compose a tidal basin of 10 acres and a closed basin of 35½ acres, divided by a space about 900 feet in width, in which there will be three large graving docks and an entrance lock. The whole of the river front of the site is enclosed, during construction, by a cofferdam, to exclude the tidal and river water. This cofferdam is more than a mile in length."

LIST OF PLANT.

Ten vertical boilers, Six 40-H.P. winding engines, Six 20-H.P. " "	Used for hauling waggons and mud scoops.
Two 40-H.P. fixed " Three 25-H.P. portable " Seven 20-H.P. " " Four 13-H.P. " "	Used for dynamos, pumps, sawmills, and other purposes, in the yard.
Four 15-inch cylinder locomotives, 6-wheeled, Four 12-inch " " 4 " Four 10-inch " " 4 " Two 9-inch " " 4 "	Used in conveying materials from landing jetties and to various parts of works.
Eight 10-ton steam cranes, Two 7-ton " " Thirty-seven 5-ton steam cranes,	Used in landing goods at jetties, lifting materials from the trenches, lowering concrete and masonry into the trenches and setting masonry, and various other purposes. Four of the 10-ton cranes are fitted to be worked as steam navvies.

* *Min. Proc. I. Mech. E.*, and *Engineering*, 28th July, 1899.

Sixteen 10-ton derrick cranes,	Used for setting masonry.
Two 10-ton Goliaths, 60-feet span,	Used for stacking granite in yard.
Ten steam winches, 8-inch cylinders,	For pile engines, &c.
Four gas engines,	For concrete mixers.
One oil ,,	For concrete mixers.
Four gas ,,	For workshops and yard.

Six dynamos.
Forty Wells' lamps.
Five rock drills, "Larmuth."
Four ,, "Little Hercules."
Two air compressors.
Six Taylor concrete mixers.
Two Sissons and White pile drivers.
Four Ruston and Procter steam navvies.
Two Baxter stone breakers.
Eight Hone grabs; four other grabs.
Seven patent mud scoops.
One 18-inch duplex pump.
One 18-inch large rocker pump.
Three 12-inch centrifugal pumps.
Two 10-inch ,, ,,
Sixteen 6-inch to 8-inch direct-acting pumps.
Two tugs, 500 I.H.P.
,, 300 ,,
Two suction dredgers; suction pipe 22 inches in diameter.
Two 800-ton steam hopper barges.
Six 1,250-ton ordinary ,, ,,
Twelve small barges of various sizes.

CHAPTER IV.

MATERIALS.

CONCRETE — THE AGGREGATE — THE MATRIX — PORTLAND CEMENT — ITS FINENESS, STRENGTH, RATE OF SETTING, AND SOUNDNESS—ADULTERANTS OF CEMENT—PROPORTION OF WATER—ACTION OF SEA WATER UPON CONCRETE—CASE OF DISINTEGRATION AT ABERDEEN—OFFICIAL EXPLANATION AND POSSIBLE CAUSES—DR. MICHAËLIS ON CEMENT IN SEA WATER—SUGGESTED PROTECTIVE MEASURES—PRACTICAL NOTES ON MIXING CONCRETE—STRENGTH OF CONCRETE—SAMPLE COMPOSITIONS—IRON AND STEEL—ALLOYS WITH MANGANESE AND NICKEL—IMPURITIES—VARIETIES OF CAST IRON, WROUGHT IRON, AND STEEL—DEFECTS IN MANUFACTURED IRON—SPECIFICATIONS FOR CASTINGS, PLATES, AND BARS—WORKING STRENGTH—TESTS—WEIGHTS—CORROSION OF IRON AND STEEL—EFFECT OF SEA WATER ON DOCK GATES—PRESERVATIVE AGENTS—TIMBER—VARIETIES USED FOR DOCK WORK—SELECTION OF TIMBER—DESTRUCTION AND DECAY—MEANS OF PRESERVATION—STONE—KINDS EMPLOYED—DESTRUCTIVE AGENCIES.

THE dock engineer has to deal with a great variety of materials common to many other branches of constructive work, and the bulk of the information requisite for a thorough appreciation of their respective qualities and uses must naturally be sought in treatises dealing exclusively with such matters. At the same time, there are other materials not so commonly employed, and there are applications, adaptations, and standards peculiarly characteristic of dock work, and it is mainly with a view of treating these special features that the following notes have been compiled. In order to maintain some continuity of form, however, it will be necessary to touch upon each subject in its general aspect, but this will be done in the lightest possible manner, and details will be reserved for those questions more particularly germane to the province of maritime engineering.

The materials selected will be dealt with in the following order:—

Concrete.	*Timber.*
Iron and Steel.	*Stone.*

CONCRETE.

Concrete is the term applied to an admixture of various mineral substances which become incorporated into a solid body under chemical action. It consists essentially of two parts—the aggregate and the matrix.

The *aggregate* is a heterogeneous mass of one or any number of the following materials:—Slag, shingle, burnt clay or earthenware, broken stone, broken brick, gravel and sand, mixed in varying proportions.

The *matrix* consists of hydraulic lime or cement, combined with water.

The above definition and classification do not include three compositions, commonly called concrete, but which differ fundamentally therefrom in that no chemical action is required to solidify them. Apart from this, their use in constructive work is very limited, and they are quite unimportant. The compositions are as follows :—

Tar concrete, made of broken stones and tar.

Iron concrete, composed of iron turnings, asphalte, bitumen, and pitch ; and

Lead concrete, which consists of broken bricks immersed in lead.

Reverting to the first and most prevalent conception of concrete, we will discuss its composition a little more in detail.

The aggregate should be clean and perfectly free from impurities, such as dust, dirt, and greasy matter. Ballast, therefore, which has been carried as such by a ship should not be used. The material should also be sharp and contain as many angular fragments as possible. Rough, porous surfaces are better adapted for the adherence of the matrix than those which are smooth and vitreous. Hence brick and gravel offer certain advantages over shingle and flints, though these latter are often preferred for a reason given below. Fragments of different size should be employed, so that the smaller material may fill up the interstices in the larger, and it is to be noted in this connection that equal measures of large and small stone, when combined, make less than double the volume of either. No individual fragment should have a dimension exceeding 4 inches, and the material is often specified to pass through a ring of $1\frac{1}{2}$ inches diameter. Weight is a desirable feature in dock walls, and accordingly for this class of work preference should be given to aggregates of high specific gravity. The amount of sand and cement will evidently be governed by the volume of the remaining cavities to be filled. These may be estimated from the following table, quoted from Mr. Sandeman's paper on "Portland Cement and Concrete" :—*

TABLE VI.

	Weight of Material.	Ratio of Interstices.
	Lbs. per cubic foot.	Per cent.
1. Broken limestone, the greater part of which would be gauged by a 3-inch ring,	95	50·9
2. Gravel, screened free from sand, varying in size between small pebbles and pieces gauged by a $2\frac{1}{2}$-inch ring,	$111\frac{1}{2}$	33·6
3. The above limestone and gravel, well mixed in equal proportions,	$113\frac{1}{2}$	33·6
4. Sandstone varying in size between pieces gauged by a 4-inch ring and pieces gauged by an 8-inch ring,	74	50·0
5. Sandstone varying in size between sand and pieces gauged by a 4-inch ring,	92	34·0
6. The above sandstones mixed in equal proportions,	$91\frac{1}{4}$	36·0

* *Min. Proc. Inst. C.E.*, vol. cxxi.

Mr. Morrison * recommends the following procedure, which, he states, he has always found a safe rule :—

"Decide tentatively on quantity of large and small stones, if necessary trying two or three proportions. Add sand by degrees, till the mixture, after being well turned over and shaken down, shows a decided increase in bulk, at least 5 per cent; then add cement to an amount equal to between one-third and one-half of the sand, and draw up a specification taking the amount of sand as unity."

A proportion of 2 parts of sand to 1 of cement will be found most effective for marine work, and it should be noted that the mortar made from sand and cement diminishes by one-fourth of the volume of the same materials mixed dry. The quantity of mortar should be from 10 to 15 per cent. in excess of that required to just fill the interstices.

The *sand* should not be too fine or dust-like, and the particles should not be rendered too spherical by attrition. Hence pit sand is better than river or shore sand.

The **matrix** is almost universally Portland cement. Hydraulic lime and Roman cement are also employed, but the range of their application is restricted. The former is useful for the foundations of buildings and the latter in cases of urgency, such as sometimes occur in tide work. Both are much inferior to Portland cement in strength and durability.

Portland cement is an artificial product obtained by calcining clay, or shale, with chalk, or other limestone, at a high temperature. It is outside the province of the dock engineer to inquire into systems of manufacture, of which there are several, or to investigate too closely the chemical composition of the cement he uses. Chemical analysis takes no account of the degree of calcination and fails to distinguish between free and combined lime.

It certainly does become necessary to acquire some knowledge of the constituents of cement in their relation to sea-water, but this question will be considered later, and, for the present, the following may be stated as the approximate composition of an average sample of sound cement :—

Lime,	60 per cent.
Silica,	23 ,,
Alumina,	7 ,,
Oxide of iron,	4 ,,
Sulphuric acid,	·5 ,,
Alkalies,	·5 ,,
Magnesia,	1·5 ,,
Moisture,	3·5 ,,
	100

Of the above ingredients, sulphur and magnesia are objectionable in excess of limits which, in the former case, are about 1, and in the latter, about 5 per cent.

* Morrison on "Cement Concrete," *Min. Proc. Inst. C.E.*, vol. cxxxix.

From the point of view of the user, the matter of greatest moment is the actual behaviour of the cement under the projected conditions. Hence the attention of engineers has been largely directed to a determination of those features which are of vital importance. The experience gained by means of numerous experiments has resulted in the selection of four characteristics for purposes of comparison, viz. :—

1. Fineness of grinding.
2. Resistance to stress.
3. Rate of setting.
4. Integrity or soundness.

Fineness.—The importance of fineness is due to the fact that the coarse particles of a badly-ground cement hydrate more gradually than the finer particles, and, consequently, expand at a later stage to the detriment of the work. Fine cement, again, will take more sand than coarse cement and makes a proportionately stronger concrete. It also possesses greater capability of rendering the concrete watertight, which under certain conditions is imperative. Finally, the coarse particles are denser and add considerably and needlessly to the cost of carriage. Fineness is tested by sieves with meshes ranging from 1,600 to 32,000 holes to the square inch, of which standards the former is as extremely low as the latter is inordinately high. General practice at present seems to favour either a residue not exceeding 5 per cent. on a 2,500 mesh sieve, or a residue not exceeding 10 per cent. on a 5,800 mesh sieve.

One caution is needful: a finely-ground cement may be obtained by supplying the mills with comparatively soft "clinker," which is inferior to that which is heavily burnt. Also, there is a point at which any increase in the fineness of the cement causes additional expense without compensating advantages. To prevent the use of light, underburnt clinker, the weight or the specific gravity of the cement is often specified. The former lies between 100 and 120 lbs. per bushel and the latter between 3 and 3·15, the higher values corresponding to the better samples, provided that the coarse particles (which have a high density) be sifted before weighing. A very heavy cement, however, is likely to contain an excess of lime, which, in the free state, is eminently deleterious. The weight, moreover, is not a very satisfactory criterion; cements decrease in weight as they grow old—as much as 4 per cent. in the first month, with a total of 15 per cent. for the year.

Strength.—With good Portland cement, mixed neat, a tensile strength of 500 lbs. per square inch should be obtained at the end of 28 days after mixing—1 day in air and 27 immersed in water. Very frequently a strength of 450 lbs. is required at the end of 7 days, but a 7 days' test is a somewhat unreliable guide to the strength ultimately attained, as cements showing but moderate results (say, 350 to 400 lbs.) at the end of a week

often develop the highest ultimate values. Uniformity of results is a great desideratum. Considerable divergency in the results is a most unsatisfactory feature, no matter how high the average may stand. It should not fail to be noted that the care taken in the preparation of the specimen briquette, and the method of applying the testing weight, exercise a very considerable influence on the results obtained.

In Germany, much importance is attached to a test in which the cement is mixed with standard sand, on the ground that the cementitious power of the cement can only be estimated properly on this basis. Indeed, it has been found that of two samples of cement, one finely and the other coarsely ground, the finer cement was the weaker of the two in the neat condition, but much the stronger in combination with sand. The test has also been introduced into this country, not with any unanimity of approval. It is difficult to procure a standard sand of rigid uniformity, and the efficiency of the test suffers in consequence. The criterion usually adopted is passage through a 400-mesh sieve and retention by a 900-mesh sieve. A briquette made with 3 parts of such sand to 1 of cement should exhibit a tensile strength of, at least, 150 lbs. per square inch at the end of a week, with a regular increase, as the period is lengthened, to 250 lbs. at the end of a month.

Compressive tests are also used in Germany, and not without reason, for concrete is particularly designed to withstand compression, whilst its use in positions of tension is strictly prohibited. The ratio of the compressive strength of Portland cement to its tensile strength may be taken at about 10 to 1. The only objection urged against this course apparently is that "Portland cement will bear a greater (compressive) stress, without fracture, than it can be subjected to in practice."*—an argument which, like the boomerang, has a curiously reflex action. It may pertinently be asked wherein the distinction lies, that the statement is inapplicable to tensile stress. The author is of opinion that an extensive series of experimental results in compression would be a very valuable addition to our data on Portland cement.

Time of Setting.—The time of setting for ordinary cement, under normal conditions, will vary between two and five hours. Slowness in setting is, generally speaking, indicative of strength. A quick-setting cement probably contains an excess of clay, but fine grinding has also an appreciable effect in accelerating the setting action, in some instances to such an extent as to justify special retardative measures. The usual way is to thoroughly aerate the cement by spreading it over a floor, under cover, to "cool," by which means the aluminate of lime becomes partially hydrated and its activity moderated. Sulphate of lime or gypsum, added to the cement during manufacture, retards the setting action, but any excess over 2 per cent. is harmful. Common soda accelerates hardening, though it weakens the

* Shaw on "Portland Cement," *Min. Proc. L.E.S.*, vol. xix.

cement.* Bicarbonate of soda, on the other hand, retards it considerably, as also do sugar, glycerine, and salt, slightly.

Integrity or Soundness.—This may be tested by Faija's steaming apparatus or by simple immersion in water. The former is the more rapid method, occupying about as many hours as the other occupies days. In both cases thin pats are made, ⅛ inch thick at the centre and as thin as possible at the edges. Signs of cracking, blowing, or expansion indicate a cement either unsound or too hot for use.

Adulterants of Cement.—Two common adulterants of Portland cement are furnace slag and Kentish ragstone, the introduction of which, though defended by some manufacturers, must be held a reprehensible practice. The first, besides being injuriously impregnated with sulphur, possesses scarcely any hydraulic properties whatever, and the second is an inferior variety of carbonate of lime. Effervescence under the action of hydrochloric acid will betray the ragstone. The slag, which is a crude mixture of silicates of lime, iron, &c., has a high specific gravity, and confers a mauve tint upon the powdered cement.

The *water* may be either salt or fresh, unless for important surface work above ground, in which case salinity is objectionable, on account of the resulting efflorescence. The amount of water required cannot be stated with exactitude. It will depend upon the proportion of the aggregate and its porosity. It is best determined by experience in each particular case. Without being profuse enough to drown the concrete or wash away the cement, it should be used in sufficient quantity to act as an efficient intermediary between the matrix and the aggregate. Some authorities advocate a very sparing use, but the author's experience is to the effect that a plentiful supply is advantageous, for several reasons: it serves to intimately incorporate the materials; if the aggregate be very porous it prevents undue absorption of moisture from the matrix, and it allows a scum of inert or limey cement to rise to the surface and pass away with the drainage. In certain parts, such as the floors and walls of graving docks, founded on water-bearing strata, and sea piers, impermeability of the work is essential to its stability, and it has been claimed by some† that a minimum of water in mixing produces a maximum of watertightness in the mixture, but this is far from being the case, and the labour involved in manipulating the concrete under such conditions is greatly increased, since, in order to secure the complete penetration of the scanty allowance of water, the mixture has to be beaten in a manner such as would be adopted to cause moisture to appear on the surface of damp sand. For the majority of dock walls, in

* Mr. F. E. Priest, of Liverpool, has been kind enough to communicate to the author the results of some experiments which he undertook in reference to this point, from which it appears that the weakening is only a transitory feature, and that at the end of four years the testing of the briquettes indicated perfectly normal results.

† *Vide* Deacon on "Liverpool Waterworks," *Min. Proc. Inst. C.E.*, vol. cxxvi., pp. 42, 43.

which impermeability is not essential, the excessive time and labour required for such an operation would be wasteful and unremunerative; and, further, there is absolutely no reason to believe that concrete mixed with a good supply of water is any the less impervious on that account. Available testimony rather demonstrates the contrary, and the following experiment* of Mr. Bamber, F.I.C., is both interesting and instructive.

He made three sets of blocks of concrete, in duplicate, with the following proportions:—4 parts of shingle, 2 of sand, and 1 of cement. The first pair were mixed with the full quantity of water that the cement would take up, which proved to be 10 lbs. for each block. The second were mixed with only 7½ lbs. of water, or three-fourths of the full quantity. The third pair were mixed with 5 lbs. of water or half the full quantity. After standing for a fortnight, one of each of these pairs was placed on a sea wall, and they were covered and uncovered by each tide. They stood there twelve months, and at the end of that time were brought on land and carefully broken through the middle. The results were as follows:—No 1, with the full quantity of water (10 lbs.) was very hard and perfectly sound and dry quite through to the surface. No. 2, with three quarters of the full quantity of water (7½ lbs.) was dry in the middle, but, on every side, the water had penetrated about 3 inches, and had much weakened the block. No. 3, with half the full quantity of water (5 lbs.) was wet quite through, and was very easily broken up, the water having been able to percolate continually through the block, and having dissolved much of the lime. The fellow pair of each of these was placed in fresh water, and remained the same time, with exactly similar results as to penetration of water and strength of blocks, but in these cases another result could be observed. In the case of No. 1, with the full quantity of water (10 lbs.), the water in which it stood remained clear. In the case of No. 2 (7½ lbs. of water) the water in which it stood became milky and turbid from the formation of carbonate of lime. In the case of No. 3 (5 lbs. of water) the water became quite white; and, at the end of twelve months, the whole block was covered with crystals, a quarter to half an inch in thickness. The lime had been gradually dissolved and crystallised on the surface in the form of calcium carbonate. Similar blocks subsequently exposed in the sea wall for nearly three years gave the same results.

Action of Sea-Water upon Concrete.—A great deal of discussion has arisen, and many conflicting opinions have been expressed, in reference to the durability of cement concrete in submarine situations. On the one hand, there are those who hold, with much practical exemplification, that concrete is in general a thoroughly reliable and durable material for use under such and, indeed, any normal conditions; and, on the other hand, there are those who point to the indubitable evidence of deterioration manifested in several well-known instances. It is a somewhat difficult matter to decide with any finality whether these failures are due to purely

* Bamber on "Portland Cement," *Min. Proc. Inst. C.E.*, vol. cvii.

local conditions, or whether they arise from causes of a more general and widespread nature. The writer has seen to the construction of a good deal of concrete work, executed without any special precautions, the whole of which during a number of ensuing years has been exposed either to constant immersion or to tidal alternations, without the slightest sign whatever of deterioration. Indeed, from specimens which have been cut out of the solid mass, he is convinced that a harder and more compact material for its purpose would be difficult to find. At the same time, the evidence in favour of adopting certain measures, of the nature of preventives against possible degeneration, is so weighty and backed by such influential authority that it cannot be lightly disregarded or passed over without due consideration.

In order then to present some evidence bearing on the question, a typical instance will be taken in which concrete, composed of Portland cement and a mineral aggregate, has proved abortive and exhibited undoubted signs of disintegration and decay.

The case is that of the entrance walls of a graving dock, at Aberdeen, opened in 1885. They were built as a "homogeneous mass of concrete, deposited inside frames, composed of 1 of cement, 2 of sand, and 3 of stone, for one-third of the depth of the frame, and of 1 of cement, 3 of sand, and 4 of stone, in the upper two-thirds." It had also been intended to provide the wall with a facing of 2 of cement, 3 of sand, and 4 of stone, but this was omitted and the surface was plastered instead. The sand used was clean, sharp, quartzose sand, screened through a sieve of 40 meshes to the inch,* and containing a small proportion of minute, water-worn pebbles. The stones consisted of smooth water-worn pebbles, granite, trap or whinstone, macadam, and granite chips.

Shortly after the opening of the dock symptoms of disruption appeared, and in June, 1887, Mr. Wm. Smith, the engineer at that period, reported that "the Portland cement concrete entrance walls have expanded $2\frac{3}{4}$ inches on the height of the walls, their surfaces have cracked and bulged, and the joints of the caisson quoin stones have opened up, causing considerable leakage."

Professor Brazier, of Aberdeen University, Mr. P. J. Messent, M. Inst. C.E., and Mr. Pattinson, a Public Analyst, were consulted on the subject.

The first-named reported as follows :—

"The analyses of the series of decomposed cements show a remarkable difference to the original cement, inasmuch as that in all these samples there is found a large quantity of magnesia, and a large proportion of the lime in the form of carbonate. I believe this alteration is brought about entirely by the action of sea-water upon the cement. There is no other source for either the magnesia or the carbonic acid."

* Although not specifically stated, apparently the linear inch is intended, and accordingly there would be 1,600 meshes to the square inch, the more generally accepted unit.

ANALYSES OF SAMPLES OF CEMENT.

	Original Cement of Test Briquette.	Decomposed Cement.				
		I.	II.	III.	IV.	V.
Alumina and oxide of iron,	13·10	26·76	28·42	1·05	1·53	5·60
Silica, ...	20·92	18·04	19·55	1·33	1·31	10·87
Carbonate of lime, . .	8·18	6·61	15·78	45·72	35·42	38·37
Hydrate of lime, . .	11·26	30·54	16·94	27·85	17·17	19·21
Caustic lime, . . .	45·39	...	...	...	...	...
Magnesia, . . .	0·33	...	...	...	...	...
Hydrate of magnesia, .	...	13·57	15·08	21·03	39·96	22·30
Sulphuric acid, . .	0·82	2·98	4·23	1·31	0·90	0·85
Soluble in water, . .	...	1·50	...	1·71	3·71	2·80

Mr. Pattinson's report, based on a séparate series of samples, contained the following conclusions :—

"On comparing the analyses of the concrete used in the work with those of the original briquettes, it is evident that very considerable changes have occurred in the composition of the cement used in the concrete. 1st. Much of the lime has disappeared from six samples. 2nd. A great increase of the magnesia has taken place in the same samples. 3rd. An increase in the amount of sulphuric acid has taken place in the same samples. This sulphuric acid exists as hydrated sulphate of lime.

"There can be no doubt, I think, that this deterioration is caused by the action of the sea water with which the cement has come in contact. According to Thorpe and Morton's analysis,* 1,000 grains of sea water contains 3·151 grains of chloride of magnesium and 2·066 grains of sulphate of magnesia. The magnesia of both these salts is precipitated as hydrate of magnesia on coming into contact with lime, with the simultaneous formation of soluble chloride of calcium and partially soluble sulphate of lime. This chemical action of sea water has evidently taken place in the cement used in the six samples, and notably in one of them, from which about two-thirds of the lime has been removed, and in which about twenty times the original quantity of magnesia, and more than three times the original quantity of sulphate of lime, have been deposited, thereby causing the friable and disintegrated condition which marked this sample. The same result, in a lesser degree, is observable in the other samples."

Mr. Messent, commenting on these analyses, observed :—"In their exam. ination of the deteriorated concrete, both agree that the presence of too much magnesia in the cement is the cause of the deterioration, and that, as the same proportion or quantity was not found in the briquettes made of the neat cement used, the additional quantity found in the spoiled concrete must have been supplied by the sea-water, in contact with the

* *Chem. Soc. Journ.*, vol. xxiv., p. 506.

cement portion of the concrete, which sea-water, whilst precipitating the magnesia that it contains, takes away, in an altered form, a portion of the lime from the cement."

Mr. Messent made experiments as to the quantity of water absorbed by briquettes of neat cement, and of cement and sand, and found that by repeated absorption and drying, the solids contained in the sea-water were left in the briquettes, the strength of which decreased by from 37 to 70 per cent.

He went on, in his report, to add:—"I am of opinion that the cause of the damage referred to is the injurious effect of sea-water, which entered through holes in the plaster, . . . percolated the concrete of the intermediate portion of the wing walls, and of the mass behind the altars of the dock walls, and, in so percolating, extracted lime from, and deposited magnesia in, the cement portion of the concrete, causing it to deteriorate and expand; and that the injurious percolation was facilitated by the inappropriate relative proportions of the cement, sand, and stone, or the insufficient quantity of cement in the original composition of the deteriorated concrete."

So much for one side of the question. The unanimity of conclusion is apparently convincing, but, at the same time, it must be admitted that other solutions of the problem are equally admissible.

In the first place, there are one or two inconsistencies in the individual reports which call for notice. Mr. Pattinson asserts that much of the lime has disappeared from his samples—as much as two-thirds in one case—while an examination of the analytical tables of Professor Brazier demonstrates as remarkable an increase in that constituent. These statements are, of course, not necessarily conflicting. The lime may have been washed away by tidal action from Mr. Pattinson's specimens, but the uniformity of its absence is curious and striking. Then, no explanation is offered to account for the very singular fluctuations, both above and below the normal quantity, of the amount of alumina and oxide of iron. A decrease is intelligible, but there is no manifest source of supply for an increase.* Aluminium salts are not present in ordinary sea-water, nor is oxide of iron a common constituent.

Without personal knowledge of the facts and circumstances, it is difficult to express a definite opinion, but it occurs to the author to suggest—

1. That the cement actually used in the construction of those portions of the wall in which decay occurred might have been of different composition, and of inferior quality, to that of the original test briquette.

2. That the aggregate was impregnated with impurities of an argillaceous nature.

(One or other of these hypotheses would appear necessary to account for the large increase of alumina in some of the specimens of decomposed

* Increase by difference in ratio is not supported by an examination of the tables.

cement, and the second would also admit of an explanation for a decrease by reason of fluxion.)

3. That the sand was much too fine for the purpose of making concrete, and was used in excessive quantities. A 1,600-mesh sieve for sifting sand is absurdly fine. In confirmation of this view, Mr. Messent's report may be quoted, in which it is said that "the deterioration was chiefly confined (so far as could be ascertained by examination) to the concrete which contained the largest proportion of sand—viz., 3 to 1 and upwards." Supplementary evidence is afforded by Mr. Leedham White,* who stated that—

"Twenty years ago he was in Aberdeen, and examined one of the concrete blocks made at the beginning of that particular work. The block was pointed out to him as not giving satisfaction to the engineers; and, although it had been made several weeks, he had no difficulty in crumbling a piece off in his hands, part of which he took home and washed, which disclosed that the sand, which had been used very liberally, was so minute in the grain that, though sharp and clean, it was little better than dust. He was so impressed with the faulty character of the sand that he took a sample of the cement to the manufacturer, and told him that he would certainly hear complaints of the cement, and ought to know how it had been treated. He did not know whether sand of that quality was subsequently used in the work, but, as a manufacturer, he affirmed that if such sand was used at the Aberdeen works during successive years, it was a miracle that the concrete had ever stood at all."

Mr. Faija,† one of the greatest authorities on the subject of Portland cement, expressed himself as follows:—

"Magnesia, as precipitated from sea-water, was simply in the form of a hydrate or carbonate, and was a perfectly inert material. The lime was dissolved from the cement, and the magnesia precipitated from the sea-water; but the lime was not dissolved to the destruction of the cement if it was sound, and, as the lime from the outside surface was dissolved, a crust of lime and magnesia was formed which rendered the mass impervious to further destructive action. He had passed sea-water through blocks under a head of 21 feet and found that, after a time, percolation ceased, because the pores of the concrete became filled with the deposit of carbonate of lime and magnesia, so that the briquettes through which the sea-water had percolated were stronger than those left in the sea-water without percolation. The analyses given by Mr. Smith showed that the failure at Aberdeen was due to bad cement or bad manipulation."

Mr. Carey,‡ who has also largely contributed to the scientific data of Portland cement, summed up the matter as follows:—

"The real point at issue is whether the salts of magnesia, which are admittedly deposited from the sea in porous concrete structures, are, or

* *Min. Proc. Inst. C.E.*, vol. cvii., p. 109. † *Ibid.*, vol. cvii., p. 118.

‡ Carey on "Portland Cement," *Min. Proc. Inst. C.E.*, vol. cvii.

are not, inert. In his opinion no conclusive evidence has been adduced to prove that the precipitates from sea-water induce disintegration, even of fissured or porous concrete, when sound cement is used. Had such evidence been forthcoming it would throw doubts on the durability of all such structures in the sea. In the Aberdeen experiments it was demonstrated that free caustic lime had been washed out of the concrete, and magnesia, as magnesium hydrate, precipitated, with the formation of calcium chloride and sulphate. The analyses prove nothing beyond the fact that the caustic lime present was the cause of such precipitation, and that the lime in this form is an unstable and soluble body. The inference, that by similar action long continued a dangerous portion of the lime may be dissolved out of the cement present in a concrete structure, is without proof. The precipitation of magnesian or other salts from sea-water is merely the deposition, without active chemical change and consequent change of volume, of bodies which already exist there in solution. Summing up the facts, of which undoubted evidence has been produced, it may be stated that an excess of caustic lime or caustic magnesia causes (1) disintegration by the expansion due to hydration; and (2) being soluble, when conditions permit of their washing out, they leave the concrete in a honeycombed state."

It would be impossible to close so vexed a question without a quotation of the views of that eminent specialist, Dr. Wilhelm Michaelis, of Berlin. He states his opinion that—

"The magnesia,* which is deposited during the action of sea-water upon hydraulic mortar, is a preservative agent which tends to close the pores of the mass. It would be more correct to speak of the injurious action of the sulphates in sea-water, than to attribute such action to the magnesia salts, although it is true that magnesium sulphate is the special salt which acts in sea-water. The sulphates of lime or of alkalies, in fact, any soluble sulphate have the same destructive action, but do not act with the same degree of energy."

"The main points † to be considered in erecting permanent structures in sea-water, with the aid of hydraulic cements—in other words, concrete—are—

"1. From the physical point of view, completely impermeable mixtures should be made, composed of one part of cement with two or, at the most, two and a-half parts of sand of mixed grain, of which at least one-third must be very fine sand. To this the requisite quantity of gravel and ballast should be added. This impermeable layer should surround the porous kernel on all sides in sufficient thickness, even underneath. It would, perhaps, be unnecessary waste of material in the case of thick walls to use the impermeable mixture throughout; but, so far as possible, the

* Michaelis on "Sea-water and Hydraulic Cements," *Min. Proc. Inst. C.E.*, vol. cxxix.

† Michaelis on "Portland Cement in Sea-water," *Min. Proc. Inst. C.E.*, vol. cvii.

compact shell and the poorer kernel should be made in one operation. Where this is not possible, and the shell is added subsequently, numerous iron ties should be used.

"2. From the chemical point of view, cements or hydraulic limes, rich in silica and as poor as possible in alumina and ferric oxide, should be used, for aluminate and ferrate of lime are not only decomposed and softened rapidly by sea-water, but they also give rise to the formation of double compounds, which in their turn destroy the cohesion of the mass by producing cracks, fissures, and bulges. The salts contained in sea-water, especially the sulphates, are the most dangerous enemies of hydraulic cements. The lime is either dissolved and carried off by the salts, and the mortar thus loosened, or the sulphuric acid forms with it crystalline compounds as basic sulphate of lime, alumino-sulphate and ferro-sulphate of lime, which are segregated forcibly in the mortar, together with a large quantity of water of crystallisation, and a consequent increase in volume results. The separation of hydrate of magnesia is only the visible but completely innocuous sign of these processes. The magnesia does not in any way enter into an injurious reaction with silica, alumina, or ferric oxide, it is only displaced by the lime from its solution in the shape of a flocculent, slimy hydrate, which may be rather useful in stopping the pores, but can never cause any strain or expansion, even if it subsequently absorbed carbonic acid. The carbonic acid, whether derived from air or water, assists the hydraulic cement as a preservative wherever it comes into contact with the solid mortar. It could only loosen the latter if present in such an excess that bicarbonate of lime might be formed.

"3. The use of substances which render the mortar, at any rate in its external layers, denser and more capable of resistance. Such substances are—

"(α) *Sesquicarbonate of Ammonia* (from gas liquor) in all cases where long exposure to the air is impossible. Such a solution applied with the brush, or as a spray, and then allowed to dry, converts the hydrate of lime into carbonate of lime. The latter is not acted upon by the neutral sulphates present in sea-water. It must be repeated that it is a decidedly erroneous opinion that the texture of otherwise sound cements is injured by the action of carbonic acid; on the contrary, it renders them more capable of resistance, except in the above-mentioned case, which is extremely rare, when bicarbonate of lime is formed and goes into solution.

"(β) *Fluosilicates*, among which magnesium fluosilicate is most to be recommended. The free lime is converted into calcium fluoride and silicate of lime, and, in conjunction with the liberated hydrate of magnesia, these new products close the pores of the mortar. Both salts are sufficiently cheap to be used on a large scale.

"(γ) Last, not least, *Barium Chloride*. Two or three per cent. of the weight of the cement is dissolved in the water with which the concrete is mixed. This forms perfectly insoluble barium sulphate with the sulphates

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of the sea-water, while the magnesia remains in solution as magnesium chloride. Although in this case there can be no further closing of the pores, yet the insoluble barium sulphate, which is formed, affords some protection and does not give rise to any increase of volume (swelling). From 2 to 3 per cent. of barium chloride does not in any way diminish the strength, as has been proved by the comparative tests of English and German cements. Frequently the strength of the mortar is increased by this addition. This substance is only to be used in the external, perfectly watertight skin of concrete; in other words, in the 4 to 8-inch coating, composed of 1 cement, 1 to 2 sand, and 3 to 4 coarse gravel, flint, broken stone, &c."

Practical Notes on Mixing Concrete for Marine Work.

1. A *heavy aggregate* is desirable. If mixed by hand, the materials should be laid out on a platform of deals, in order to secure freedom from dirt and impurities, and covered by the cement in a thin layer. The whole should be turned over thrice dry, and as many times wet, before depositing.

2. The concrete should not be tipped from a height greater than 6 feet, or there will be a tendency for the heavier portions of the aggregate to separate from the lighter. For great depths, shoots may be employed with men stationed at the foot to shovel the mass immediately into position. The work should be well rammed and consolidated.

3. As many *rubble burrs*, or *stone plums*, should be imbedded as the fluid concrete can adequately enclose. No two burrs should be in contact, and none should be set within 12 inches of the face of the wall. If the burrs are porous, they should be wetted before insertion.

4. The concrete should be deposited without delay after mixing, and should remain entirely undisturbed during setting. After the setting of each layer, the surface should be prepared for the reception of the next layer by picking, washing, and sweeping. In mass work, layers should not exceed 2 to 4 feet in height.

5. *Concrete blocks* should not be used under 14 days after mixing, and preferably the period will be extended to three or four weeks.

6. *Concrete bags* have a tendency to break away at the ends. Consequently, they should be slightly longer than the nett length required.

7. *Wind screens* should be provided in windy weather, otherwise the cement will be largely wasted, even if the concrete be not allowed to suffer thereby.

8. Concrete mixing should be avoided as far as possible during keen frost, except in situations where the concrete is deposited directly under water, or is soon afterwards covered by the tide. Where continuous operations are essential on shore, artificial warmth from braziers and fires may be utilised to raise the surrounding temperature, and salt-water may be employed in mixing on account of its lower freezing point. An American

practice is to dissolve 1 lb. of salt in 18 gallons of water when the temperature is 32° F., and to add 3 ounces for every 3° of lower temperature. The surface of such work, left for the night, must be protected by boards, tarpaulins, sacking, gravel, or littered straw.

Strength of Concrete.

Compressive Strength.—The following results were obtained by Mr. Grant.* Experiments were undertaken with 12-inch cubes of compact concrete made with Portland cement, weighing 110·56 lbs. per bushel, and having a tensile stress of 427 lbs. per square inch after seven days' immersion in water. The tests took place at the end of twelve months.

TABLE VII.

Composition of Concrete.	Crushing Weight in Tons.	
	Blocks kept in Air.	Blocks kept in Water.
1 cement, 1 ballast,	107	170·5
1 ,, 2 ,,	149	160
1 ,, 3 ,,	113	115·5
1 ,, 4 ,,	103	108·5
1 ,, 5 ,,	89	99·5
1 ,, 6 ,,	80·5	91
1 ,, 7 ,,	75	80·5
1 ,, 8 ,,	61·5	76
1 ,, 9 ,,	54	68·5
1 ,, 10 ,,	48·5	48

Experiments made with 9-inch cubes of the concrete (6 of gravel and broken stone to 1 of Portland cement) used in the construction of the Vyrnwy Dam gave 84·23 tons per square foot as the lowest resistance to compression in the case of a block little more than three months old, and 298·6 tons per square foot as the highest resistance in the case of a block three years old. The mean resistance to cracking, under compression, of all the blocks tested between two and three years after moulding was 215·6 tons. Still higher results were obtained from blocks cut out of the hearting of the actual work. The mean resistance to cracking, under compression, of 19 blocks, between one and two years old, was 263 tons per square foot.

Transverse Strength.—In an experiment by Mr. Colson † a beam of 9 to 1 concrete, 28 days old, 21 inches wide, 9 inches deep, and 3 feet 9 inches clear span, fractured with a weight of 1·044 ton applied centrally. The coefficient derived from this, for the unit beam, 1 foot wide, 1 foot deep, and 1 foot span, becomes 4 tons.

* Grant on "Strength of Portland Cement," *Min. Proc. Inst. C.E.*, vol. xxxii.

† *Min. Proc. Inst. C.E.*, vol. liv., p. 270.

In an experiment by Mr. Sutcliffe with a concrete block cut from a dock wall at Liverpool, and composed of 8 parts of gravel and broken brick to 1 of Portland cement, with rubble burrs incorporated, the size of the block being 25 inches wide by 23 inches deep, and the clear span 12 feet, fracture resulted from a central concentrated load of 3·25 tons, giving a coefficient of 5 tons for the unit beam.

Sir Benjamin Baker's experiments,* in which the weight of the beam itself was included, yielded the following unit breaking weights:—

4·85 tons for	. . .	8 to 1 concrete.
9 ,,	. . .	6 to 1 ,,
13 ,,	. . .	4 to 1 ,,
18 ,,	. . .	pure cement.

Some Sample Compositions of Concrete.

1. At Arbroath, used by Mr. W. Dyce Cay, in 1887, for a dock entrance—
   1 Portland cement, 7 sand, gravel, and broken stone.
2. At Sydney, used by Mr. C. W. Young, in 1883, for a graving dock—
   1 Portland cement, 1·5 sand,
   3·61 bluestone, gauged through a 2½-inch ring.
3. At Belfast, by Mr. W. Redfern Kelly, in 1888, for a graving dock.
   (*a*) For foundations in tideways—
   1 Portland cement, 1½ gravel,
   2 sand, 1½ whinstone metal.
   (*b*) For hearting to walls—
   1 Portland cement, 2½ whinstone metal,
   2 sand, 3½ coarse gravel.
   (*c*) For facing to walls—
   1 Portland cement, 3 fine gravel.
   1 sharp sand,
4. At Newport, Mon., by Mr. G. D. Pickwell, in 1889, for a graving dock—
   1 Portland cement,
   10 broken steel slag, weighing 26 feet per ton, in pieces not larger than 2½-inch cubes for bulk and ¾-inch cubes for face work—in both cases unscreened from dust.
5. At Greenock, by Mr. W. R. Kinipple, between 1878-86, for dock walls—
   1 Portland cement, 3 ballast,
   3 coarse sand, 6 large stones.
6. At Ardrossan, by Mr. R. Robertson, *circa* 1889, for dock walls.
   (*a*) For rubble concrete—
   1 Portland cement, 1·4 gravel,
   2 broken stone, passed through screen with 2-inch mesh. 2·2 sand,
   (*b*) For concrete in bags—
   1 Portland cement, 1·4 gravel,
   2·2 broken stone, 1·2 sand.

* *Min. Proc. Inst. C.E.*, vol. cxi., p. 95.

IRON AND STEEL.

Cast iron, wrought iron, and steel are essentially the same substance in combination with different proportions of other constituents. The principal ingredient in this connection is carbon, and the following percentages are generally recognised as forming the distinctive compositions of the three classes of metal, viz. :—

From ·0 to ·1 per cent for wrought iron.
,, ·3 ,, 1·8 ,, ,, steel.
,, 2·0 ,, 5·0 ,, ,, cast iron.

Unfortunately, this quantitative differentiation is not susceptible of too strict interpretation, because other ingredients, besides carbon, exercise a powerful modifying influence upon the compounds. Their properties also depend upon the form in which the carbon is present—whether as specks of graphite, or free carbon, mechanically mixed and easily detected, or in such intimate chemical combination as to be indistinguishable from the metal itself.

A practical distinction is founded upon the behaviour of a bar of metal under certain treatment, as follows :—

Steel attains great hardness when suddenly cooled, from a high temperature, by immersion in water or oil. This process has no effect upon wrought iron.

Steel which has been hardened in this way may be softened again, or tempered, by heating it and allowing it to cool gradually. Cast iron may be hardened, but it cannot be tempered.

One drawback to the efficacy of these tests is that some modern steels, containing elements other than carbon and iron, are made softer, and not harder, by sudden cooling.

A third attempt at drawing a distinction relies upon the results obtained in the testing machine, but this method is too artificial to be of any practical value.

Altogether, it must be confessed that, while the differences in the physical properties of iron and steel are sufficiently marked to preclude any misconception, it is no easy matter to lay down any definite line of demarcation between the metals themselves. Steels containing less than ·5 per cent. of carbon form an intermediate class insensibly shading into, and gradually acquiring the characteristics of, wrought iron. Such steels are commonly designated *mild steels*, and they furnish the bulk of the material used for structural purposes. Those compounds containing a higher percentage than 1·5 imperceptibly merge into the class of cast irons.

The influence exerted by carbon in modifying the physical characteristics of iron, while largely dependent upon the manner in which it enters into combination with it, may be stated in general terms as follows :—

A relatively large proportion of carbon induces hardness, strength, incompressibility, brittleness, and fusibility. A small proportion tends to toughness, malleability, weldability, and tenacity.

Manganese Steel.—But, as already remarked, there are other constituents, besides carbon, which are capable of entering very largely into combination with iron, and of exercising an influence equally powerful in determining its characteristics. By far the most remarkable is an element which, according to the proportion in which it is incorporated, imparts the most opposite qualities to the compound. The addition of manganese to iron was suggested as far back as the 18th century,* and Mushet, who published in 1840 the results of some very interesting experiments, recommended it as an essential accompaniment to the Bessemer process. The quantity recommended was small and in the form of *spiegeleisen*, and a limit was found at which the steel apparently ceased to benefit by the admixture. A recent and more deeply experimental investigation, by Mr. R. A. Hadfield, has established the important fact that there is a second limit beyond the first, at which the deterioration ceases, and the compound commences to regain in greater intensity the characteristics which it had seemingly lost. Mr. Hadfield's conclusions are as follows:—†

"Whilst the belief, hitherto held, that steel becomes brittle and comparatively worthless when the manganese exceeds about 2·75 per cent. is correct, yet it has now been proved that, by adding more of the same metal in such quantities as to obtain in the material under treatment not less than about 7 per cent. of manganese, the result is a metal with entirely different characteristics; in fact, the product is a new metal. The apparent paradox thus takes place that, whilst manganese, alloyed with iron, the former being present in the proportion of not less than 2·75 and up to about 7 per cent., gives a very brittle product, when its proportion is increased to not less than 7 and up to about 20 per cent., . . . the result is a material possessing peculiar and extraordinary strength, toughness, and other qualities."

Manganese steel is more free from blow holes than are ordinary castings, and the addition of silicon, in order to prevent unsoundness or honeycombs, is unnecessary. Whilst molten, it gives off a peculiarly strong sulphurous odour, and, though at first very fluid, it cools more rapidly than ordinary steel; its contraction is also greater.

Nickel is a second agent capable of entering into an effective combination with iron, and of producing a valuable compound. The following

* Early experiments upon manganese were made by Glauber in 1656, and by Wartz in 1705. Rinman (1773) melted equal parts of grey pig-iron and manganese ore, obtaining a non-magnetic product. Reynolds attempted its use in the manufacture of steel in 1799.

† Hadfield on "Manganese Steel," *Min. Proc. Inst. C.E.*, vol. xciii.

concise statement of its influence is given by Mr. White, of the Bethlehem Iron and Steel Co., U.S.A. :— *

"The tensile strength and elastic limit of nickel iron alloys and nickel steel rise with increasing proportions of nickel, reaching a maximum at about 20 per cent. Passing this they begin to fall, and elongation increases abnormally up to 30 per cent. The hardening effect of quenching ceases at about 10 per cent., but is quite marked in the lower percentages. In this case the effect is heightened by the manganese, but with ·06 per cent. manganese it is still decided. Between 10 and 20 per cent. nickel, neither quenching nor annealing exerts any decided effect. Above 20 per cent., quenching produces a softening effect, which is decided at 30 per cent. Perhaps it would be better to call it a weakening effect, as the tensile strength and elastic limit are much lowered, the elongation increased, but the cutting properties shown by turning in a lathe are not perceptibly changed.

"These results refer to alloys of nickel and iron containing carbon from ·06 to ·1 per cent., which practically can be considered carbonless alloys, as it is impracticable to make them lower. The manganese ranged between ·06 and ·1 per cent.

"There are many difficulties to be overcome in handling nickel steel as commercially made. It is very susceptible to changes of temperature when containing the usual amounts (·2 to ·9 per cent.) of carbon and manganese, requiring considerable care in heating and working to bring out its best qualities."

The question of alloys is a very wide one, and, in view of the extensive range of modern chemical research, the student will do well to consult technical literature for a more complete and detailed statement of the behaviour of the various products. It would be out of place here to enter into the subject seriously, and we must dismiss other known combinations with the briefest possible notice.

Tungsten and *chromium* have the effect of hardening steel and increasing its tenacity. *Copper* and *antimony*, on the other hand, produce brittleness. *Titanium* increases the ductility.

The following constituents are usually accounted impurities :—

Silicon produces brittleness in iron and is generally excluded as slag. It is not detrimental to steel when present in a very minute quantity, as it tends to repress agitation and bubbling during the process of cooling. Its effect on cast iron is somewhat similar to that of carbon.

Phosphorus hardens cast iron, makes it more fusible, and lessens its ductility. Steel is deteriorated by a very small quantity, say, ·08 per cent. Wrought iron is rendered more weldable by ·01 per cent., but above that limit the metal becomes brittle and "cold short"—*i.e.*, it cracks if bent cold.

Sulphur makes wrought iron "red short," or brittle, at high temperatures. It renders both steel and cast iron more fusible and more brittle.

* *Min. Proc. Inst. C.E.*, vol. cxxxviii., p. 53.

Classification of Iron.—A description of the various processes employed in the manufacture of iron and steel is quite beyond the scope of the present work. A brief classification of mercantile products, with their most noteworthy features, is all that can be attempted.

Pig iron is the name given to the coarse bars of unpurified metal run off from the blast furnace. These are roughly divisible into two kinds—those having a dark grey fracture, due to a large proportion of uncombined carbon, and those having a silvery fracture, with very little uncombined carbon. The first are distinguished as *foundry pigs*, being particularly suitable for castings, and the second as *forge pigs*, being only adapted for conversion into wrought iron. Special varieties of pig are generally assigned to the manufacture of steel. For what is known as the acid process (see below), the metal must be comparatively free from phosphorus and sulphur, such, for instance, as the pig produced from hæmatite ores. By the basic process much impurer ores, containing a large proportion of phosphorus, can be utilised, but the product is scarcely so satisfactory.

Cast iron is obtained by remelting pig iron to eliminate its impurities. The process may be repeated with beneficial results as many as a dozen times. After that point has been reached the metal begins to deteriorate. According to Sir William Fairbairn, the transverse strength and elasticity decrease after the twelfth remelting, and the compressive strength after the fourteenth. Cast iron comprises three classes—grey, mottled, and white cast iron, following the structural nature of the pigs from which they are cast. The first contains a profusion of carbon in graphitic specks, the last is free from uncombined carbon, and the second represents an intermediate condition.

Chilled iron is a product of casting in which the surface of the metal is allowed to come into contact with a cold substance, with the result that it becomes hard and brittle while the interior remains tough.

Wrought iron is iron from which all carbon has been eliminated as far as practicable. It is developed in a pasty mass which is much improved by cutting, piling, and rolling. Hence there are three qualities, each an amelioration on the preceding by a repetition of the process—viz., *puddled bars*, *merchant bars*, and *best bars*.

Steel is capable of production on two systems (1) by eliminating the carbon from pig iron until the requisite proportion is left, and (2) by adding a definite amount of carbon to wrought iron.

The cementation process based on the second system produces, first, *blister steel* of very unequal quality, and secondly, *shear steel*, in which the metal is rendered more homogeneous by piling and rolling. *Cast steel* is obtained by melting, in crucibles, wrought iron which has been previously bedded in charcoal powder in a furnace.

The *Bessemer* process yields a steel of that name, which is due to the combustion of the carbon contained in suitable pig iron, by means of a volume of air forced at high pressure through the molten mass, leaving

the iron either at the exact composition required, or comparatively pure, so that the requisite carbon may be added to it.

Siemens-Martin steel results from the reduction of a mass of crude iron, often with the admixture of an ore rich in oxide, the whole being melted in an open hearth exposed to the intense heat of a regenerative furnace. It is a much slower process than the Bessemer, but it produces a steel of a more generally trustworthy character, and it is frequently specified for bridgework.

Of the above processes, two modifications exist—viz., (*a*) the Acid, and (*b*) the Basic—according to whether the lining of the converters, or of the furnaces is siliceous or calcareous. In the basic process, additions of calcined lime are made to the bath of molten metal in order that it may combine with the excess of phosphorus, and remove it in the form of slag. In the acid process this step is not taken, and hence the necessity for purer ores.

Practical Observations on Manufactured Iron.

Defects in Castings.—The engineer should have sufficient acquaintance with foundry methods to enable him to appreciate the difficulties of successful casting, to understand the proper distribution of the metal for the purpose intended, and to distinguish between defects which are trifling and those which are of vital importance. Founders incur considerable risks and many failures in their endeavours to reproduce large and intricate patterns, and such work should not lightly be rejected on account of some insignificant surface blemish, when otherwise sound and serviceable. On the other hand, there are surface indications, apparently slight, which reveal serious internal defects.

The necessity for having the metal thoroughly fluid, in order that it may penetrate to all parts of a large mould, sometimes causes it to be heated to such an extent that it burns into the sand of the mould, and instead of producing the clear blue skin of the ideal casting, a rough white surface is the result. This affects green sand moulds rather than those of loam or dry sand.

The most common defects of castings are the presence of *blow* or *air holes* due to the generation of steam and gases by the damp sand, the want of sufficient venting, and an imperfect supply of metal. A certain amount of dampness in the sand of moulds and cores is necessary to secure adhesion of the particles, but an excess of moisture produces steam. An insufficient number of vents causes particles of air to be imprisoned in the various parts, and an imperfect supply of molten metal lacks the head to secure homogeneity. Very often these blowholes are not manifest until the casting is machined, and occasionally they escape notice altogether. It is obvious that they are a source of weakness wherever they occur. The author has noticed a hydraulic pressure pipe develop an almost imperceptibly fine jet through the thickest part of the flange, while the

thinner stem remained intact. To remedy such defects in large pieces, without having recourse to a fresh casting, the hole may, under certain circumstances, be bored, tapped, and fitted with a steel screw, or a wrought iron patch may be raised to a white heat and hammered in. A sound casting is, however, always preferable to one that has been doctored up.

Another defect is the presence of extraneous matter, such as loose sand from the mould, or even impurities in the iron itself. These latter should be skimmed off the surface of the casting ladle. But it is difficult to avoid loose sand in a mould which takes some time to close, and light projections are frequently washed away by the influx of metal. Such foreign matter will naturally rise to the top of the casting, and by making the latter a little higher than the nett size required the objectionable material can be removed later by the planing machine.

Imperfectly adjusted cores cause the metal to be thicker on one side of a hollow casting than on the other. While perfect adjustment is perhaps not always attainable, yet limits of deviation should be fixed and adhered to.

Shortage in the supply of metal to a mould cannot be made good by a second charge. No matter how quickly applied, a shut or flaw will be the inevitable result.

Castings which become cracked or twisted are frequently due to defective design. Considerable variation in the thickness of the metal, abrupt changes, and outlying projections cause irregular contraction. The thinner portions cool more quickly than the thicker portions, and internal stresses, often unsuspected, are set up. Sudden changes in sectional area should, accordingly, be avoided, and projections should be graduated from the main body.

Specification for Castings.

"Castings are to be clean, true, and free from twist, having regular surfaces both inside and outside, with sharp, well-defined angles and lines. They must be sound and free from air or sandholes, cold shuts, and other imperfections. In the case of columns, pipes, &c., care should be taken that the lengths are exactly equal to the dimensions given; that the bearing surfaces of flanges are perfectly smooth and regular planes, perpendicular to the centre line; that the bolt holes are of the proper size and in their exact positions, and that the thickness of the metal in the shafts is quite uniform throughout, of which evidence may be taken by drilling holes, if necessary. Any casting will be liable to rejection which deviates more than $\frac{1}{16}$ inch in thickness and $\frac{1}{4}$ inch in length from the given dimensions."

Defects in Rolled Plates and Bars.—Loose and open fibres, flaws, and signs of lamination are due to imperfect rolling and welding.

Coarse crystals or blotches of colour are caused by an insufficiently purified metal, contaminated with *scoriæ* and other impurities.

A crystalline fracture does not necessarily imply an inferior iron. When

wrought iron breaks suddenly a crystalline fracture is the invariable result. A truer test is a slowly applied breaking weight, which should cause a fibrous fracture. Bad iron is never fibrous.

Specification for Plates and Bars.

"Every plate and bar must be sound, straight, and free from all flaws, and any piece which shows signs of lamination or other defect will be rejected. The edges of all plates are to be planed so that they may bear truly at their joints. All joggles are to be thoroughly well and neatly formed. The butting ends of all ties, angles, and bars are to bear fairly and firmly throughout, and all corners and edges to be neatly finished off. Every piece is to be of the full thickness specified, to be tested by gauging, weighing, or otherwise."

Working Strength.—The following table gives the amount of stress generally permissible, in tons, per square inch of sectional area:—

TABLE VIII.

	Cast Iron.	Steel.	Wrought Iron.
Tension, ...	$1\frac{1}{2}$	8	5
Compression,*	8	12	4
Shearing, ...	2	6	4

These figures are based on a factor of safety of 4. The Board of Trade has fixed the limit of stress for bridges of wrought iron at 5 tons per square inch, and of steel bridges at $6\frac{1}{2}$ tons. The strength of steel depends on the precise nature of its composition, and the values given above are merely approximate and general.

Tests.—*Cast iron* is usually specified to be tested as follows:—A sample bar is cast, 3 feet 6 inches long, 2 inches deep, and 1 inch wide. It is supported on bearings 3 feet apart, and loaded at the centre with a weight variously stated at from 25 to 30 cwts., which it is required to sustain without fracture and without exhibiting a deflection greater than $\frac{5}{16}$ inch. Test bars should, if possible, be cut from the casting, but in any case should be cast under exactly the same conditions. A tensile test is rarely required.

Wrought iron is generally required to stand a minimum tensile stress before breaking, the contraction of area at fracture not being less than a

* These values only apply in the case of short struts. When the length is considerable, failure is more likely to take place through flexure, and special calculations are necessary for determining the nature and extent of the stress. The problem is dealt with in Chapter ix.

certain amount. According to the quality desired the following figures are given:—

	Ultimate Stress.	Contraction.
Round or square bars, . .	23 to 27 tons.	20 to 45 per cent.
Flat bars, ...	22 to 26 ,,	16 to 40 ,,
Angle or tee iron, . . .	21 to 25 ,,	12 to 30 ,,
Plates with grain lengthways, .	20 to 24 ,,	8 to 20 ,,
Plates with grain crossways, .	17 to 22 ,,	3 to 12 ,,

In addition to this, certain forge tests are required. Thus, 1-inch plates for the Admiralty are to be capable of bending without fracture while hot from 90° to 125° along the grain and from 60° to 90° across the grain, and while cold, 10° to 15° along the grain and 5° across the grain. For $\frac{1}{4}$-inch plates the cold tests are 55° to 70° and 20° to 30° respectively.

Steel, according to Admiralty requirements, must have an ultimate tensile strength of between 26 and 30 tons per square inch, combined with an elongation of 20 per cent. in a length of 8 inches. Lloyd's specification raises the limits to between 27 and 31 tons with the same elongation. Both tests apply, indifferently, along or across the grain.

As regards temper, strips cut from a plate heated to a low cherry-red and cooled in water at 82° F. must stand bending round a curve of which the diameter is $1\frac{1}{2}$ or 3 times the thickness of the plate, according as the authority is Lloyd's or the Admiralty.

Rivets, if of wrought iron, should be capable of being bent double, cold, without sign of fracture. When hot they should stand being hammered down to less than $\frac{1}{8}$ inch in thickness without cracking at the edge. If of steel they should have an elongation of 25 per cent., with 26 to 28 tons per square inch tensile strength, in test pieces of ten diameters, and should be capable of bending double after the same tempering as that applied to steel plates.

Weight of Iron and Steel.—Plates of metal, 12 inches square and 1 inch in thickness, weigh $37\frac{1}{2}$, 40, and $40\frac{3}{4}$ lbs. respectively for cast iron, wrought iron, and steel.

Corrosion of Iron and Steel.—It is to be regretted that on a point of such vital importance to the dock engineer as the durability of metal structures exposed to atmospheric and aqueous agencies, the evidence is so scanty as to be inconsiderable, so incomplete as to be inconclusive, and so conflicting as to be actually perplexing. This state of things arises from a variety of causes. In the first place, it is only within the last fifty years that iron has begun to usurp the pre-eminence hitherto enjoyed by wood and stone in maritime construction, and steel is an intrusion of still later date. Consequently there has hardly yet been sufficient time in which to acquire data for the determination of the actual life of metallic structures

under such conditions, even if systematic experiments had been carried out from the earliest possible moment, which has not been the case. Again the variation in atmospheric conditions is extremely great, the seasons being marked by enormous fluctuations in sunshine, rainfall, and temperature not only for different seasons in the same year, but for the same season in consecutive years. The question is still further complicated by the factor of locality. Then, as regards aqueous influences, there is no definite standard of comparison whatever. The salinity, acidity, density, and temperature differ in almost every unit volume of sea-water, so that it is never precisely the same at any two ports. Rivers, sewers, and ocean currents all contribute to differentiate its composition.

It would, perhaps, be a comparatively easy solution of the difficulty to lay down one's individual experience as a dogma for general acceptance, but the wiser and more judicious course will be to set forth such information on the subject as is available, and leave the reader to draw his own conclusions.

The following coefficients, given by Thwaite and quoted by Molesworth,* represent the amount of corrosion in lbs. per square foot of surface during twelve months' exposure :—

TABLE IX.

Material.	Corroding Agents.					
	Foul Sea-Water.	Clear Sea-Water.	Foul River-Water.	Pure Air or Clear River-Water.	City Air or Sea Air.	Sea-Water of Average Foulness.
Cast iron, ...	·0656	·0635	·0381	·0113	·0476	...
Wrought iron, ...	·1956	·1285	·1440	·0123	·1254	...
Steel, ...	·1944	·0970	·1133	·0125	·1252	...
Cast iron (skin removed by planing),	·2301	·0888	·0728	·0109	·0884	...
„ (surface galvanised), . .	·0895	·0359	·0371	·0048	·0199	...
„ in contact with brass, ...						·1908
„ „ „ copper, ...						·2003
„ „ „ gun-metal, ...						·3493
Best wrought iron in contact with brass, ...						·2779
„ „ „ copper, ...						·4012
„ „ „ gun-metal, ...						·4537

If the metal be painted once a year the coefficient to be divided by 2; if once in two years, by 1·8; and if once in three years, by 1·6.

Trautwine † states, in apparent contradiction of the above, that while "fresh-water corrodes wrought iron more rapidly than cast, the reverse appears to be the case with sea-water," and that "the corrosion of iron or steel by sea-water increases with the carbon." He admits, however, that

* *Pocket-book of Engineering Formulæ*, 25th edition, p. 33.

† *Civil Engineers' Pocket-book*, 17th edition, p. 218.

wrought iron is affected very quickly, so that thick flakes may be detached from it with ease. The following instances are cited :—"Cast-iron cannons from a vessel which had been sunk in the fresh-water of the Delaware river for more than 40 years, were perfectly free from rust." The cast-iron work of the "Royal George" and the "Edgar," sunk in the sea for 62 years and 133 years respectively, when examined by Gen. Pasley had become quite soft and resembled plumbago. The wrought iron was not so much injured, except when in contact with copper, or brass gun-metal.

Two other experimentalists—Rennie and Mallet—adopt antithetical opinions as to the relative corrosion of cast and wrought iron in salt-water. The former maintains a higher rate for cast iron; the latter, for wrought iron.

The following table extracted from a paper on the corrosion of iron and steel, by Mr. David Phillips,* relates to a series of experiments made by him with five sets of iron and steel plates, 4 inches square by $\frac{3}{8}$ inch thick, exposed to various corrosive agencies. "To avoid even a suspicion that galvanic action had any influence in these cases, all the plates were suspended on glass rods, and each plate was separated from its neighbour by glass ferrules." It is important to note that Mr. Phillips attributed the generally greater corrosion during the first period of trial to the fact that the weather in the summer of 1879 was much more changeable than that in 1880.

TABLE X.—Corrosion of Iron and Steel.

Metal.	Water.	Loss of Weight.			
		First 12 Months.	Second 12 Months.	Total.	Average per Sq. Ft. of Surface.
		Grs.	Grs.	Grs.	Grs.
N Bessemer steel, .	Rain-water, . .	186·7	141·4	328·1	1,246·9
Y Siemens steel, .	,, . .	174·1	147·0	321·1	1,220·3
B B Staffordshire iron,	,, . .	165·3	119·0	284·3	1,080·5
D D Yorkshire iron, .	,, . .	185·1	136·2	321·3	1,221·1
N Bessemer steel, .	Sea-water, . .	42·4	36·9	79·3	301·4
Y Siemens steel, .	,, . .	33·5	34·7	68·2	259·2
B B Staffordshire iron,	,, . .	35·4	35·6	71·0	269·8
D D Yorkshire iron, .	,, . .	36·9	31·6	68·5	260·3
N Bessemer steel, .	Exposed to weather and dipped in sea-water daily, .	1,044·7	501·6	1,545·6	5,874·0
B B Staffordshire iron,		417·9	259·1	677·0	2,572·9
Y Siemens steel, .	Exposed to the weather only, .	234·4	135·9	370·3	1.407·3
D D Yorkshire iron, .		147·6	52·7	200·0	761·2

In the discussion which followed the reading of the paper, much emphasis was laid by Dr. Siemens, Mr. Barnaby, Mr. Farquharson, and others, on the importance of removing the magnetic oxide scale from the

* Phillips on "The Comparative Endurance of Iron and Steel when Exposed to Corrosive Influences," *Min. Proc. Inst. C.E.*, vol. lxv.

surface of steel, and this received the confirmation of Sir W. H. White, at a later meeting of the institution, when he declared that "as regards the relative corrosion of iron and steel when immersed in sea-water, the experience of the Admiralty during the last six years (1876-1882) showed that if the manufacturers' scale (black oxide) was thoroughly removed, and equal care taken in protecting the surfaces by paint or composition, iron and steel had about the same average rate of corrosion, the steel wearing somewhat more uniformly than the iron."*

The question of corrosion principally concerns the dock engineer in regard to the duration and maintenance of metal gates and fittings. Decay mainly takes place below the water-line, where inspection and repairs are alike difficult. In this connection the following data taken from a report † by Messrs. Brandt and Hotopp to the Ninth International Navigation Congress possess much interest :—

"I. In the case of the floodgates at Glückstadt, erected in 1874 and to be renewed this year (1902), the first isolated rust spots on the outer skin are to be found at 4 inches below ordinary low water level; the spots increase in number at 6 inches below low water, and are thickly distributed all over the metal at a depth of 10 inches. The greatest depth to which decay has penetrated in the strip comprised between this line and another, lying about 3 feet 3 inches below low water, is about $\frac{1}{4}$-inch; below this level the metal skin is covered with a layer of short-stalked moss, mixed with shells, the thickness of which increases downwards, and below which the depth and extent of decay grows gradually less and less (to about $\frac{1}{8}$-inch deep near the sill), so that the plates near the sill are almost sound. A few of the rivet heads, starting at a depth of 14 inches below low water, begin to show signs of decay and are furrowed; the decay gradually increases with the depth, so that when the rows of rivets, situated between 18 and 22 inches below low water, are reached, not only have all their heads been completely eaten off, but their shanks have also been already attacked in isolated cases. The decay in this case also becomes less and less with increased depth. The water of the River Elbe, at Glückstadt, is only on exceptional occasions somewhat brackish, but in the outer harbour there is a great deal of deposit, and several drains full of water from the moors empty into it.

"II. The gates, and more especially the floodgates, in the harbour at Geestemünde, erected in 1861, show a furrow, the rust in places penetrating as deep as $\frac{3}{10}$ inch into the outer metal skin, just above the cover strips lying close below low-water line, and it may be assumed that similar rusty places exist also above the cover strips in lower situations, the upper portions of the outside rivet heads lying close under low water mark have also rusted away. The cause to which this damage is ascribed is the layer

* *Min. Proc. Inst. C.E.*, vol. lxix., p. 35.

† Brandt and Hotopp on "Iron, Steel, and Wooden Gates," *Int. Nav. Cong., Düsseldorf*, 1902.

of mud deposited on the upper edges of the cover strips and on the rivet-heads, the mud being highly charged with acids derived from the decaying river deposit and the salt-water and water from the moors conveyed by the lower Weser and the Geste. The corrosive influence of the deposit is proved by the fact that the decay in question is specially noticeable on the convex side of the curved floodgates, the outer skin of which is permanently immersed in the very muddy water of the outer harbour, whereas on their concave side they are often washed by the water in the harbour which is not so turbid.

"III. The dock gates of the new harbour at Bremerhaven were erected in 1852, and removed as worn out in 1900. The thinning down of the plate was especially noticeable where projecting edges formed ledges upon which mud could settle. Those parts of the gates which had been in contact with oak timber were also in worse condition. At Bremerhaven the water is fairly full of salt and heavily laden with mud.

"IV. The inner gates of the great lock at Harburg, erected in 1880 and removed in 1901 for alteration, were found in very good condition with the exception of a strip about 2 feet wide near the low water-line, where the outer skin was very rough and showed rust spots penetrating $\frac{1}{8}$ inch into the metal. The river-water is completely free from salt and almost free from mud at Harburg, but the water in the harbour is, as yet, strongly polluted by the surface and house drainage of the town, and several chemical factories, besides, discharge their waste water into it full of impurities, the oxidation of all which takes place on the surface of the water; consequently, the plating of the gates is principally damaged near the water-line."

The following statement of results, obtained by the author in some experiments, covering a period of twelve months, serves to illustrate the difficulty of deducing reliable coefficients of corrosion from any but the most extensive investigation. The data obtained are not without intrinsic interest, but in order to be of any practical value, such observations would have to be extended over a considerable number of years. It is a note-worthy feature that the galvanised specimens apparently suffered more than the ungalvanised, and that, during the first three months, the latter, instead of losing, actually gained, weight. This is due partly to the conditions of immersion, and partly to the fact that weight is, after all, no very reliable criterion of the amount of corrosion actually taking place, since some forms of oxidation involve no loss in this respect.

The first six specimens were suspended in a disused clough-shaft, to which the tidal water of the River Mersey had free access, the specimens being placed at mean tide level, so that they were in and out of water for about equal periods. The water was somewhat impregnated with sewage discharged from a neighbouring outfall sewer, and the ungalvanised specimens became coated with a hard deposit, apparently of a calcareous nature, which was removed as far as possible before each weighing by washing in

clear water and using a stiff scrubbing brush. The gain in weight of certain of the specimens represents the amount of deposit which could not be removed in this way. No further measures were taken to remove the deposit, because it was deemed desirable to maintain the normal conditions of corrosion.

The last three specimens were kept continuously immersed in the water of an inner dock, which was free from contamination.

Precautions were taken to prevent any contact between the various pieces, and all were well washed prior to each weighing.*

TABLE XI.

Nature of Specimen.	Nett Area of Surface exposed.	Initial Weight.	Weight at end of 1 Month.	Weight at end of 3 Months.	Weight at end of 6 Months.	Weight at end of 12 Months.	Total Loss in 12 Months	Loss per sq. in. of exposed Surface.
	Sq. ins.	Grains.	Grains.	Grains.	Grains.	Grains.	Grains.	Grains.
Casting, plain, . .	37·12	19,473	19,499	19,526	19,510	19,457	16	·43
,, galvanised, .	36·87	18,272	18,228	18,170	18,197	18,069	203	5·50
Wrought-iron bar, plain,	54·96	31,126	31,175	31,173	31,167	31,083	43	·78
,, galvanised,	54·38	31,437	31,351	31,282	31,278	31,243	194	3·56
,, turned, .	58·35	31,172	31,187	31,185	31,143	30,988	184	3·15
Steel bar, plain, . .	23·89	6,662	6,672	6,685	6,671	6,633	29	1·21
Cast-iron plate, . .	77·5	16,979	16,976	16,965	16,943	16,972	7	·09
Wrought-iron plate, .	75·25	12,903	12,883	12,838	12,814	12,854	49	·65
Mild steel plate, . .	75·25	13,519	13,503	13,483	13,453	13,406	113	1·50

It may be useful, as well as interesting, to insert here an analysis of the water of the River Mersey, made by Mr. Charles C. Moore, F.I.C., in September, 1897. The sample was taken about the time of high water, and its specific gravity was found to be 1·02254. The water contained the following salts in solution :—

Sodium chloride, ...	22·35	grammes per litre.
Sodium bromide, ...	0·32	,, ,,
Potassium chloride, ...	0·54	,, ,,
Magnesium chloride, ...	2·78	,, ,,
Magnesium sulphate, ...	1·785	,, ,,
Calcium sulphate, ...	1·9	,, ,,
Calcium carbonate, ...	0·04	,, ,,
Total dissolved salts, . . .	29·715	,, ,,

Preservation of Iron and Steel.—The two principal measures adopted for preventing corrosion are painting and galvanising.

Painting is an operation which should be repeated, at least, once in three years under normal conditions, and oftener in exposed situations.

* In regard to this last operation, the author desires to acknowledge the kind assistance he received from Messrs. H. Pooley & Son, Ltd.

As a general rule, lead paints* are employed, but it has been suggested that preference should be given to oxide of iron paints, to avoid any tendency to galvanic action between two metallic substances. Care should be taken to remove all rust and scale before applying the paint.

Cast iron on leaving the mould has, or should have, a hard bluish skin, which should be kept intact by an immediate coat of (hydro-carbon) oil or paint. Wrought iron is also sometimes specified to be dipped in oil while hot, but the method is not a very successful preservative, and ironworkers dislike it on account of its messiness.

Dock gates and other marine structures of iron and steel should be thoroughly scraped, cleaned, and painted at frequent intervals—in some cases annually. The materials usually employed for the purpose include—red lead and oil paint, mineral tar, vegetable tar, black varnish, and siderosthen. The surfaces of ironwork in close contact should be painted before being put together. The interior walls of ballast boxes, and other generally inaccessible surfaces, are frequently floated with a thick wash of Portland cement.

Galvanising consists in immersing the iron in a bath of molten zinc, whereby a skin of that metal is formed upon the surface. The process is successful so long as the zinc covering remains intact. When it cracks, or becomes defective in any way, rapid corrosion ensues in the presence of the least damp. The writer's experience of galvanised iron, employed as a material for dock sheds, is that sea air, highly charged with salt and moisture, works havoc with it. Several such sheds, after being a few years in existence, have had to be completely coated with black varnish to preserve them from imminent destruction.

The *Angus-Smith treatment*, largely adopted for cast-iron pipes, consists in dipping them, at a temperature of 700° F., into a mixture of coal tar, pitch, linseed oil, and resin, at a temperature of 300° F. The process is an admirable method of preservation, and enjoys a considerable reputation.

TIMBER.

The varieties of timber principally in demand for the purposes of dock engineering may be enumerated as follows :—

Piles.—Greenheart, Jarrah, Karri, Mora, Pitchpine, Oak, Elm, Beech.

* A very common constituent of modern paints is sulphate of barium, of which there are two forms, viz. :—(1) the finely-ground mineral barytes, and (2) *blanc fixe*, or precipitated sulphate of barium. While both these substances have the same chemical composition, there is a wide difference in their physical conditions, which results in the ground mineral being worthless as an ingredient of paint, whereas the precipitate is just as valuable, owing to its covering power and unalterability. Examination of a sample of each paint under the microscope will easily show the difference between the fragments of crystals in the first case and the amorphous condition of the other.

Gates and Cloughs.—Greenheart, Jarrah, Mora, Oak, Pitchpine, Pine, and Fir.

Deckings (for wharfs and bridges).—Greenheart, Oak, Teak, Elm.

Fenders.—Elm.

Temporary Dams.—Pitchpine.

Timbering for Excavations.—Pitchpine, Spruce deals, Greenheart sheeting piles.

Graving Dock Blocks.—Oak, Birch, Elm, Pitchpine.

As indicative of their comparative values in maritime situations, the following classification of timbers for shipbuilding purposes, by a committee of Lloyd's, will be useful:—

TABLE XII.

Estimated Durability in Years.	Timber.
12	Teak, British oak, mora, greenheart, ironbark, sal.
10	Bay mahogany, cedar.
9	European Continental oak, chestnut, blue gum, stringy bark.
8	North American white oak and chestnut.
7	Larch, hackmatack, pitchpine, English ash.
6	Cowrie, American rock elm.
5	Red pine, grey elm, black birch, spruce fir, English beech.
4	North American hemlock, pine.

Greenheart is a product of British Guiana and the north coast of South America. It is a wood of extreme hardness and durability, with a colour ranging from green to black. Its resistance to crushing is enormous, but it is very brittle and splits under the least provocation. Particularly is this the case during the months of April and May. Great care is therefore required in working it, and when a log is about to be sawn in two, it is often advisable to bind it on each side of the proposed cut with chains and wedges. The wood has a very fine grain and exhibits no distinct annual rings. It is very heavy, ranging from 62 to 75 lbs. per cubic foot, so that it does not float in water. It contains an essential oil which is very poisonous, and which apparently confers upon it some immunity from the attacks of sea-worms. The evidence on the last point, however, is not conclusive. Greenheart is obtainable in logs from 12 to 24 inches square and up to 70 feet in length.

Mora is a light red wood with similar uses to greenheart and is a native of the same district. It is very tough and close-grained, difficult to saw and split, and extremely durable. It can be obtained in logs 18 to 24 inches square and as much as 100 feet in length.

Purpleheart, another neighbouring tree, is also noted for its qualities of durability and strength. It is hard and close-grained, and its colour is

indicated by its name. Owing to its great toughness it is capable of resisting great shocks. Logs can be obtained from 18 to 30 inches square.

Bullet tree is a dark red wood said to be an excellent substitute for greenheart. It saws easily, takes a smooth finish, and is thoroughly tough and durable. The size of the logs runs up to 3 feet in quarter girth and 50 feet in length.

Kakaralli, though a less known tree, is described as even surpassing greenheart in its qualities for marine situations, such as durability and resistance to the attacks of worms. It is close-grained, tough and difficult to saw, but easy to plane. It has one drawback in that it can only be obtained in small logs, 10 to 14 inches square, and rarely exceeding 40 feet in length.

Jarrah is an Australian timber, resembling mahogany in colour, also recommended as a valuable substitute for greenheart. It is hard and close-grained, very liable to warp and split and full of clefts, filled with resinous matter. The fibres contain an acid having a pungent odour, said to be very efficacious against sea-worms and insects. Its extreme durability compared with other timbers is incontestible, and it is on record that it has survived the attacks of marine borers long after other woods have succumbed. On the other hand, there are some authenticated instances of its destruction by the white ant and the teredo.

Karri, another Australian native, is hard, heavy, straight-grained, and tough. It is stronger than jarrah but less durable in damp situations, though when entirely immersed it is said to last well. No decisive evidence is forthcoming as to its capacity to resist worms.

Red Gum is another tree possessing the same characteristics as jarrah, with strength and toughness in a higher degree, while its durability is rather less.

Ironbark is one of the hardiest and strongest woods in existence, but it is not so durable in marine situations as the preceding varieties, being admittedly readily attacked by the teredo. In spite of this fact it is much used for piles in harbour works in New South Wales. The wood has a close, straight grain, is very tough and heavy, and is white or yellowish in colour.

Blue Gum, though an undoubtedly useful timber, is only suitable for dry and open situations, and it is depreciated by a tendency to warp and shrink under exposure to the sun. It is straw-coloured.

Stringy bark is a hard, heavy, straight-grained wood, occasionally employed for the superstructure of engineering works. This concludes the Australian series.

Keyaki is a very important timber in Japan, being strong, durable, and easily worked. It is durable in situations alternately wet and dry, and is much used for piles.

Deodar, supposed to be a variety of the Cedar of Lebanon, is a wood of great stiffness, strength, hardness, and durability, well adapted for engineering purposes in India.

Sâl, or **Saul**, is a close-grained, straight-fibred wood, possessing the same useful characteristics as the deodar, but much stronger and more durable. The wood is heavy and coarse in grain.

Teak, perhaps the best known of Indian trees, is endowed with considerable strength and durability. It has been designated the Indian oak, but it is also found in Burmah, Siam, and Java. The grain is fine and straight, the wood light and easily worked, with a tendency to splinter. Teak contains an aromatic oil of a resinous nature, which coagulates to such a degree of hardness as to spoil the edges of cutting tools. The oil is further reputed to be a preservative from the white ant and from sea-worms. Marketable logs do not exceed 40 feet in length, with a quarter-girth of 15 inches downwards. Teak is mainly used in small scantlings.

Elm is a wood of great strength and toughness, found generally on the continent of Europe and in North America. The grain is smooth, close, and fibrous, offering great resistance to crushing. English elm is brownish in colour, with a curly grain not easy to split, and it is a noteworthy feature that the sapwood is equally durable with the heartwood, provided the conditions be those of total immersion or complete dryness. Alternations of wet and dry bring about speedy decay. In American elm, which is lighter in colour, stringy in the grain, and liable to split under exposure, the heartwood alone is durable, and that only when kept constantly under water.

Beech is a light, compact, fine-grained wood, grown in Europe and the United States. It is readily cleavable and easily worked. Like elm it is subject to decay under changes of condition, but is fairly durable if maintained in either the wet or the dry state.

Oak is possibly the most valuable timber of northern latitudes, and English oak is particularly renowned for its strength and toughness. It is unfortunately liable to the attacks of insects, and it contains an acid which has a tendency to corrode iron fastenings. American oak is considered somewhat inferior to the English and European varieties.

Pitchpine is a product of the Southern States of North America. It is a strong, heavy resinous wood, extremely durable, when not exposed to marine insects, but difficult to work, and subject to cupshakes. It is procurable in logs, reaching up to 70 or 75 feet in length, with 10 to 20 inches quarter-girth.

Pine, Deal, Fir, and **Spruce** are terms covering a large variety of timber of the same generic character, which it is not necessary to discuss here at any length, more especially as the wood plays no part of unique importance in dock work. Its uses are confined to purposes common to most structures. It is a very handy material, with a considerable range of strength and toughness among the various species. It will be sufficient to remark that wood from the Baltic is generally superior to that from North America. Red pine from Scandinavia makes the best timber for framing, and spruce deals from the same locality make admirable sheeting piles. The former is

imported in logs, 12 to 14 inches square, and the latter in scantlings of 9 inches by 3 inches and in lengths up to 16 feet. Oregon pine is noted for the great length and girth of its logs, but it is not a very strong or durable wood. Signs of decay have been observed in a very short time. It is, however, very useful for temporary shoring, and can be obtained from 30 to 130 feet in length with 12 to 30 inches quarter-girth. Hemlock, from the Southern States of South America, is in demand for deals and sleepers.

TABLE XIII.—Weight and Strength of Timber.

Timber.	Weight in Lbs. per Cub. Ft.	Transverse Strength in Lbs.	Timber.	Weight in Lbs. per Cub. Ft.	Transverse Strength in Lbs.
Greenheart, .	62 to 75	900 to 1,500	Elm, . .	34 to 37	350 to 450
Mora, . .	57 ,, 68	1,100 ,, 1,250	Beech, . .	43 ,, 53	550 ,, 700
Purpleheart, .	62	...	Oak, . .	49 ,, 61	500 ,, 650
Bullet tree, .	67	...	Pitchpine, .	41 ,, 58	500 ,, 700
Kakaralli, .	63	...	Birch, . .	45 ,, 49	550 ,, 650
Jarrah, . .	63 to 64	500 to 650	Fir, . .	34 ,, 36	400 ,, 700
Karri, . .	63 ,, 64	650 ,, 850	Pine, . .	32 ,, 34	350 ,, 500
Red gum, .	53 ,, 63	650 ,, 720	Spruce, . .	29 ,, 32	400 ,, 500
Ironbark, .	72	950 ,, 1,050	Chestnut, .	35 ,, 41	550 ,, 650
Blue gum, .	63 to 71	550 ,, 850	Cedar, . .	35 ,, 47	400 ,, 500
Stringy bark, .	58	450 ,, 650	Ash, . .	43 ,, 53	600 ,, 700
Teak, . .	41 to 52	600 ,, 700			

Note.—The transverse strength given above is the concentrated central breaking weight of a beam 1 inch wide, 1 inch deep, and 1 foot clear span.

Selection of Timber.—A thorough insight into the merits and defects of different logs can only be obtained by much experience and close personal investigation. The selection of timber for important marine works should, accordingly, only be entrusted to a competent and reliable man. It would be a difficult matter to enumerate all the indications of weakness in logs, and many defects are quite latent to the inexperienced eye. Shakes or splits should be looked for and their extent gauged by tapping. Discoloration is a bad sign, as also are sponginess and the appearance of wormholes on the surface. Timber with large or dead knots is unsuitable. The heart should be central. Rankine* states the following general indications of strong and durable timber:—

"In the same species, that specimen will in general be the strongest and the most durable which has grown the slowest, as shown by the narrowness of the annual rings.

"The cellular tissue, as seen in the medullary rays (when visible), should be hard and compact.

"The vascular or fibrous tissue should adhere firmly together, and should show no woolliness at a freshly cut surface, nor should it clog the teeth of the saw with loose fibres.

* *A Manual of Civil Engineering*, p. 441.

"If the wood is coloured, darkness of colour is, in general, a sign of strength and durability.

"The freshly-cut surface of the wood should be firm and shining, and should have somewhat of a translucent appearance. A dull, chalky appearance is a sign of bad timber.

"In wood of a given species, the heavier specimens are, in general, the stronger and the more lasting.

"Among resinous woods, those which have the least resin in their pores, and amongst non-resinous woods, those which have least sap or gum in them, are, in general, the strongest and most lasting."

Decay and Destruction of Timber.—Timber is subject to dry and wet rot and to the depredations of worms and insects. *Dry rot* is a disintegration of the fibres accompanied by the growth of a fungus, due to, and accelerated by, inadequate ventilation. It attacks woodwork in confined situations free from moisture, and reduces it to the condition of a fine powder. The disease is infectious, and spreads with startling rapidity. Once attacked, no remedy can save the affected parts, and the only efficient preventive is thorough ventilation. *Wet rot* is a decomposition of the fibres under the influence of moisture, resulting in putrefaction and decay. It is not infectious like dry rot, but is communicable to sound timber by actual contact.

Of worms and insects which attack timber, impair its strength, and in some cases bring about its utter destruction, the most important are the *Teredo navalis*, the *Limnoria terebrans*, the *Chelura terebrans*, and the *Termes* or white ant.

The *Teredo* is found in all British waters, and, indeed, frequents the majority of seaports. It has a preference for clear salt water, and the available evidence seems to point to the fact that it avoids fresh, sewage-polluted, and muddy water with equal impartiality. Its depredations take the form of tunnellings or excavations into the timber, generally along the grain, and these it lines with a chalky secretion. It is no uncommon experience to find holes ½ inch or ¾ inch in diameter. Some specimens of the *Teredo* are very large, measuring as much as 2 feet in length.

The *Limnoria* is a small insect, which is troublesome on account of the vast numbers in which it infests certain localities. It appears to be indifferent to the foulness of the water, provided it be saline. Its ravages are confined to the range of the tide, and it generally works about high-water level of neap tides.

The *Chelura* is a shrimp, which undercuts woodwork and causes it to fall away in flakes. This insect manifests a decided partiality for pure sea-water, and is, consequently, more often found along the open coast than in enclosed harbours.

The *Pholas dactylus*, while principally regarded as an enemy of masonry, has also been known to attack wood. It bores a number of holes close together.

Above ground, timber is subject to the depredations of ants—particularly, in tropical climates, the *white ant*. Even the hardest woods succumb to its attacks. The boring is most insidious, the whole of the interior being eaten away, while the surface remains intact.

Preservation of Timber.—Of all artificial means available for the protection of timber, alike from destruction and decay, by far the most satisfactory is the process of creosoting. It coagulates the albumen and fills the pores with an antiseptic substance, which excludes moisture, repels worms and insects, and prevents dry rot.

Creosote is an oily liquid contained in the second distillation of tar. Its composition is somewhat variable; but, in order to be effective, it should contain over 40 per cent. of naphthaline, about 4 or 5 per cent. of carbolic acid, and as little pitch as possible. The process is as follows:—The timber to be treated, after being dried, is placed in a vacuum, and there heated to vaporise the sap and expel all traces of moisture. Creosote at a temperature of about 120° F. is then introduced into the containing cylinder under considerable pressure. The liquid is absorbed by the wood to an extent ranging between 3 and 16 lbs. per cubic foot. The former figure applies to oak and other hard woods, which are rather unsuitable subjects for treatment. Soft, and even green, woods are better adapted on account of their higher power of absorption.

Other substances have been advocated for the impregnation of timber, notably solutions of sulphate of copper (Boucherie's process), corrosive sublimate (kyanizing), and chloride of zinc, but they do not give such good results as oil of tar. A Commission appointed by the Dutch Government some time ago, for the purpose of investigating the claims of various preservative agencies, reported that "the only process which could be relied upon for the protection of wood from the attacks of the *Teredo* was that of creosoting."

Apart from internal treatment, various superficial applications have been tried, with more or less success. Paint is a very usual agent and an effective preservative, provided it be applied only to well-seasoned timber and periodically renewed. If applied to green timber, it imprisons the sap and induces decay. In sea-water the coating is liable to be softened and eroded. Tar, verdigris, and paraffin have also been employed as external coverings.

The extremities of timber posts let into the ground are frequently charred to a height of a few inches above the ground level.

For open woodwork in marine situations the following measures have been adopted, with generally favourable results, more particularly in regard to the attacks of worms:—

Metallic Sheeting.—A thin covering of copper-plate has proved to be a most satisfactory protection for piles, but it must extend from below the surface of the mud to somewhat above high water mark, otherwise the insect may intrude itself between the metal and the wood. The drawback

to its extensive use is its expense. A zinc covering has been tried, but it is soon corroded by sea-water. Muntz metal is another substitute.

Pipe Casings.—Piles encased in earthenware pipes, such as drain pipes, with the intervening space filled in with sand or cement grout, make a durable combination in situations free from shocks and erosion. A coating of Portland cement will often answer the same purpose, but it is more likely to crack. Tubes of steel wire netting, embedded in concrete on the Monier principle, have been found very effective.

Compound Coverings.—The following method, used on the Pacific coast, has attracted attention:—"After removing the bark, the surface of the pile is covered with a prepared compound, some of the ingredients of which are paraffin, powdered limestone, and kaolin. The pile is then wrapped in jute burlap, and another application of the compound is made. Wooden battens are then nailed along the surface, which receives a final coat of the paint. Piles thus protected have been in use for ten years. The coating protects the piles from the teredo, limnoria, and similar animals, but its duration is not known." *

Close Nailing.—The driving in, very closely together, of broad-headed scupper-nails is an expedient of some antiquity. The heads are apt to rust, and though this is sometimes held to be a further protection from worms, the statement lacks confirmation. The method has been applied to dock gates, but it is troublesome and expensive.

A natural protection is very often provided by the accumulation of barnacles, mussels, and other shellfish upon the surface of the wood. Sea thorns act in the same way when the surface has been covered with their discs.

STONE.

There are many varieties of stone suitable for constructive work, but the dock engineer confines his attention to a comparative few, which, by long experience, have gained a reputation for durability and strength. The principal of these is granite.

Granite is a very hard and extremely durable rock, of igneous origin, crystalline in structure, and of great value in dock work on account of its heavy and massive proportions. In its true form it is composed of crystals of quartz, felspar, and mica; but there are other—so-called—granites containing hornblende (syenitic granite), quartz diorite, &c.

The quartz is a very hard substance, with a vitreous lustre, and practically indestructible. It renders the granite very difficult to work. The felspar is lustrous and granular, and, being present in greatest volume, gives the granite its distinctive colour, which may be white, grey, pink, red, or brown. It is less hard and less durable than quartz. Mica is a thin, flaky substance, with a bright, metallic lustre. It is easily decomposed.

* Snow on "Marine Woodborers," *Engineering*, Oct. 7, 1898.

Granite is principally used in situations where great strength is required, such as for copings and facings to dock walls, quoins and sills to entrances and locks, column and pivot bases, girder beds, paving setts, and road metal.

The stone is procured in various parts of the United Kingdom, chiefly in Aberdeenshire, Kirkcudbrightshire, Cornwall, Devonshire, Leicestershire, Wicklow, Wexford, and the Channel Isles. Cornish granites have generally a very coarse grain.

Sandstone has a crystalline structure composed of grains of quartz cemented together by various substances, such as carbonate of lime, carbonate of magnesia, &c., upon the weathering qualities of which the durability of the stone depends. A good sandstone should possess a uniform, compact, bright, well-cemented grain. A dull appearance is not a good sign. Some sandstones are very friable, others are but moderately durable, but a few of the harder varieties are very serviceable for dock work, such as those from the reputed quarry of Bramley Fall,* near Leeds, from the Forest of Dean, in Gloucestershire, and elsewhere.

TABLE XIV.—COMPRESSIVE STRENGTH OF STONE.†

Stone.	Crushing Weight in Tons per Square Foot.	Stone.	Crushing Weight in Tons per Square Foot.
Granite—Aberdeenshire, .	800 to 1,200	Limestone—Chilmark, .	400
Cornish, . .	600 ,, 1,000	Magnesian, .	430
Mount Sorrel, .	850	Sandstone—Craigleith, .	350
Trap—Penmaenmawr, .	1,050	York, . .	350
Limestone—Portland, .	250	Bramley Fall,	390
Bath, . .	90 to 100	Cheshire, .	130
Purbeck, .	580		

Limestone is a somewhat vague term for a stone, the principal constituent of which is carbonate of lime; and a class which includes chalk, Portland stone, Kentish rag and marble, has a very wide range of characteristics indeed. The most durable specimens, as a rule, are heavy, dense, and homogeneous, with a fine, crystalline grain. Portland and Purbeck limestones, perhaps the best known varieties in general use, differ slightly from this criterion; the first has a fairly large grain, and the second is conchoidal and non-crystalline. Both these stones, and, indeed, limestones generally, and in a lesser degree sandstones, are vulnerable under the attacks of the *Pholas*, and this acts as a deterrent to their extensive use in marine situations. The limestone blocks at Plymouth

* The original quarry of Bramley Fall is reported to be practically worked out, but much of the stone from neighbouring quarries goes by the same name.

† For a very valuable and complete series of experimental results, dealing with the crushing strength of stone, the reader is referred to a paper on "The Building Stones of Great Britain," by Professor T. Hudson Beare.—Vide *Min. Proc. Inst. C.E.*, vol. cvii.

breakwater had to be replaced by granite blocks owing to the ravages of the mollusc. Apart from this, the growing popularity and the ready adaptability of concrete have caused it to largely supersede natural rock for dock construction and harbour works.

Destruction of Stone.—The softer kinds of stone will frequently wear away under continued attrition and the chemical action of an unsuitable atmospheric environment, but the destructive agencies most in evidence, in regard to the more adamantine varieties used in dock work, are living organisms.

The *Pholas dactylus* is a mollusc, living in sea-water, which bores into limestone, shale, sandstone, and timber, but does not attack granite. It is a small animal, with a maximum length of about 5 inches, but one which is quite capable of doing extensive mischief by boring its holes in close proximity to each other, causing the ultimate collapse of the masonry.

The *Saxicava* is another mollusc known to bore into limestone to a depth of 6 inches. It has manifested its presence at Plymouth, Folkestone, and elsewhere.

There is apparently no remedy for the ravages of these marine borers, except the substitution of some other kind of material for the stone attacked.

CHAPTER V.

DOCK AND QUAY WALLS.

Definition—Functions under Various Conditions—Stresses in Retaining Walls—Overturning Forces—Angles of Repose—Theory of Conjugate Pressures—Coulomb's Theorem—Chaudy's Theorem—Weight of Earthwork—Surcharge—Restraining Forces—Counterforts—Tie Bars—Weight of Masonry—Empirical Formulæ—Conditions of Stability—Centres of Gravity—Typical Example—Practical Points—Natural Foundations—Stratified Sites—Artificial Foundations—Piling—Wells and Cylinders—General Methods of Construction, with Examples of Quay Walls at Newcastle, Cork, Glasgow, Liverpool, Belfast, Ardrossan, Marseilles, Antwerp, Rotterdam, Dublin, Kurrachee, Suez, Bougie, and Sfax—Consideration of Instances of Failure at Altona, London, Southampton, Calcutta, and Liverpool—Underpinning—Miscellaneous Types of Wall at Hull, Greenock, London, Liverpool, and Manchester.

Definition.—A dock wall may be said to be a special case of a class of walls termed Retaining or Revetment walls. Under normal conditions it derives a certain, albeit varying, amount of support from the hydrostatic pressure on its face, which more or less neutralises the earth pressure from the rear. Should, however, the dock at any time be allowed to run dry, the identity of its functions with those of an ordinary retaining wall would be complete. This is a possibility which may have to be faced, voluntarily, on account of repairs and alterations, or involuntarily, for other reasons, such as an accident to the entrance gates. Accordingly, it is advisable to neglect any frontal sustaining force and to treat a dock wall as if it were a retaining wall, pure and simple.

But, even in so doing, it must be admitted that the range of contingencies to which a dock wall is liable far exceed those affecting an ordinary retaining wall. "Hydrostatic pressure alone may more than double or halve the factor of safety in a given wall. Thus, with a well puddled dock bottom, the subsoil water in the ground at the back of the wall will frequently stand far below the level of the water in the dock, and the hydrostatic pressure may thus wholly neutralise the lateral thrust of the earth, or even reverse it. On the other hand, with a porous subsoil at a lock entrance, the back of the wall may be subjected, on a receding tide, to the full hydrostatic pressure due to the range of that tide plus the lateral pressure of the filling. Again, the water may stand at the same level on both sides of the wall, but may or may not get underneath it. If the wall is founded on rock or good clay, there is no more reason why the water

should get under the wall than that it should creep under any stratum of a well-constructed masonry or puddle dam, and under those circumstances the presence of the water will increase the stability by diminishing the lateral thrust of the filling. If, however, as is perhaps more frequently the case, the wall is founded on a porous stratum, the full hydrostatic pressure will act on the base of the wall, and reduce its stability in practical cases by about one-half." * These mutable conditions can manifestly only be met by providing a considerable margin of strength.

Stresses in Retaining Walls.—The forces at work in the case of an ordinary retaining wall are three in number:—

(1) There is the overturning influence of a wedge-shaped mass of earth, D C E (fig. 77), behind the wall, which tends to slide down some plane of rupture, C E, in the absence of proper support.

(2) To this must be added the effect of any surcharge upon the surface of the ground constituting the wedge.

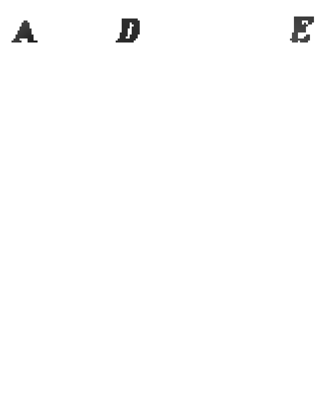


Fig. 77.

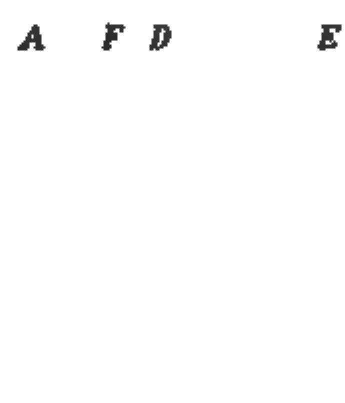


Fig. 78.

(3) And, lastly, there is the weight of the wall acting vertically downward, and consequently offering resistance to the overturning tendency. If the back of the wall be not vertical, as in fig. 78, it is obvious that the perpendicular line, C D, must still be considered the virtual boundary of the opposing influences and that the weight of the earthwork, F C D, must be included in the weight of the wall.

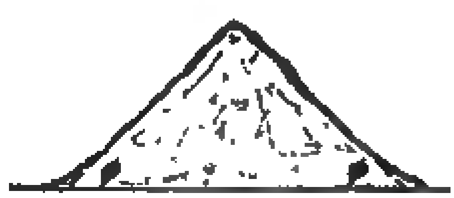


Fig. 79.

It will be well to consider these forces a little more in detail.

Overturning Forces.—The actual extent of the wedge and its effective pressure can only be matters of conjecture. It is common experience that unsupported earthwork stands at widely differing slopes, according to the nature and condition of the particles of which it is composed. To a limited degree, experiments have determined some of these slopes and fixed what is termed an *Angle of Repose* (φ, fig. 79) for the more prominent kinds of

* Baker on "Lateral Pressure of Earthwork," *Min. Proc. Inst. C.E.*, vol. lxv., p. 180.

earth. But the values attached to these angles can only be regarded as of an approximate nature, as will be evident from a glance at the following table comprising maximum and minimum results obtained by different experimentalists:—

TABLE XV.

Material.	Range of Angle of Repose.	
	From	To
Gravel and shingle,	35°	48°
Dry sand,	21°	37°
Vegetable earth,	28°	55°
Compact earth,	40°	50°
Well-drained clay,	40°	45°
Peat,	14°	45°

Ranges so extensive render it an exceedingly difficult matter to assign any angle to a variety of soil, however specific, especially in view of a further modification due to its degree of humidity. The amount of moisture present in the sample under consideration very materially influences the experimental result obtained for its angle of repose. A slight quantity, just sufficient to occupy the interstices between the grains of solid matter, has been found to increase the frictional resistance to movement, and, accordingly, to produce a correspondingly greater angle of repose. Any excess of moisture, however, over and above this trifling amount, results in a diminution of the frictional resistance; and if the humidity be indefinitely increased, the material eventually acquires a muddy consistency to which there is no angle of repose worth noting. Ordinary clay, for instance, in the dry condition crumbles at 40°; moderately moist, its inclination may be increased to as much as 50°; allowed to become saturated, it subsides at an angle of 10°.

Argillaceous earths are most susceptible to the deteriorating influences of moisture, and any admixture of sand with the clay only produces an accentuation of the evil, because the impermeability of the clay offers an obstacle to the escape of water which has entered through the pores of the sand. A striking instance of this is afforded in a notable landslip behind a quay wall at Altona, to be dealt with at a later stage.

The foregoing considerations distinctly emphasise the necessity for the prompt and adequate drainage of earthwork, and particularly so in the case of dock and river walls, where the earth backing is generally in a state of intermittent immersion. Under the head of a rising tide, water penetrates to an equal height behind the wall, and, unless there be adequate means for its withdrawal with the ebb, the volume of water thus confined will prove a serious augmentation of the overturning forces.

Quite apart, however, from the question of humidity, there is another difficulty in the way of estimating the angle of repose for cases in practice. The earth behind a dock wall is often anything but homogeneous. With the most moderate foundation depths, a series of totally different strata will generally be passed through, each having its own particular angle of repose. And even supposing the most favourable case—that of filling of a fairly uniform texture—it is manifest that the increased pressure upon the lower layers will confer upon them a greater density, and so modify their conditions of stability that the line of rupture, instead of being straight, will become more and more inclined. Further, the absence of pressure upon the topmost layers will enable these to stand at a steeper inclination, so that the natural outline of the mass would present the form of an ogee curve (fig. 80). Altogether, it must be frankly confessed that it is practically impossible to arrive at any thoroughly reliable data for dealing with each case *in situ*, and, in the absence of definite information, the only course open is to make certain assumptions, approximately accurate, and to allow a sufficient margin of safety to cover attendant errors.

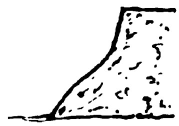


Fig. 80.

Several theories, accordingly, have been put forward in regard to the magnitude and direction of the resultant pressure of earthwork on a retaining wall. It would be impossible, within the limits of this work, to investigate all these theories exhaustively, but it will be noticed that, however distinct in development, they contain a common elemental factor.

Considering the wall as of unit length, calling the height h (A B or C D, fig. 77), and the angle of rupture θ, the sectional area of the earth wedge may be stated as $\frac{h^2 \tan \theta}{2}$, and its weight as $\frac{w h^2 \tan \theta}{2}$, w being the weight per unit volume. The various theories may then be covered by the following general expression:—

$$P = \frac{w h^2}{2} \times C, \qquad . \quad (11)$$

in which P stands for resultant pressure, and C is a variable coefficient dependent upon several considerations, such as the angle of repose, ϕ, the surface slope, α, of the earth behind the wall, the batter, β, of the back of the wall, and the direction, γ, of the resultant.

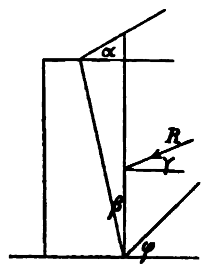


Fig. 81.

In the ensuing examination of some of these theories, the foregoing symbols will retain their respective significations throughout.

The Theory of Conjugate Pressures.—Professor Rankine, in his work on *Civil Engineering* (pp. 167 and 318), has developed a theory of earth pressure which ignores the existence of any cohesion between the particles. It is based on the following principle, primarily enunciated in a paper on

"The Stability of Loose Earth," contributed to the Philosophical Transactions of the year 1856, viz. :—"The resistance to displacement, by sliding along a given plane, in a loose granular mass, is equal to the normal pressure exerted between the parts of the mass on either side of that plane, multiplied by a specific constant." The restriction renders the theory somewhat defective in its relationship to ordinary revetment walls with well-consolidated backing, but it is nevertheless apparent that any calculations made on this basis will err only on the side of excessive strength.

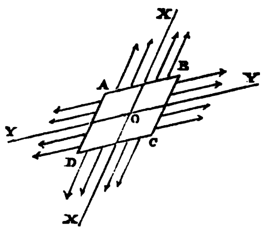


Fig. 81*a*.

Starting with a definition of conjugate stresses as a pair of stresses acting upon two planes supposed to traverse a point in a body, such that each stress is parallel to the plane upon which the other acts, and, futher, distinguishing as principal stresses those stresses which are mutually normal,* we may go on to show that there are three cases in which the intensity and direction of the resultant stress can be determined, viz. :—

1. When the principal stresses are of the same kind—*i.e.*, either both positive (compressive) or both negative (tensile), with equal intensities.

2. When, with equal intensities the stresses are not of the same kind; and,

3. When the stresses are of either kind, but with unequal intensities.

Case I.—The resultant stress must clearly be of the same kind as the principal stresses, and have an intensity equal to that of either of them. In fig. 82, A B and B C are planes upon which two principal stresses, P and Q, are supposed to act. Since these are, by hypothesis, equal in intensity, heir magnitudes will be proportional to the sides, A B and B C, respectively. If, then, from the point of intersection we set off O X to represent $P = p \times AB$, and O Y to represent $Q = q$ (or p) $\times$ B C, O Z will give the magnitude and direction of the resultant, R. Since the triangles, A B C and O X Z, are similar, it follows that R is perpendicular to the plane, A C, and is proportional to the side, A C (*i.e.*, $R = r \times AC$), and, therefore, that the intensity of pressure of the resultant is equal to the intensity of each of the principal stresses, which is equivalent to stating that $r = p = q$.

Case II.—When the sense of one of the principal stresses is altered, the intensities remaining equal, the effect is to change the direction of the resultant, but not its amount or intensity. In fig. 83 the principal stresses are P and Q, as before, but the sense of P is inverted. By a construction

* If two planes, X X and Y Y, be supposed to traverse a point, O, in any body, and if the direction of the stress, p, on the plane X X be parallel to the plane Y Y, then the direction of the stress, q, on the plane Y Y is parallel to the plane X X, and the two stresses are said to be conjugate. When X X and Y Y are at right angles the stresses become principal stresses (fig. 81a).

similar to that in Case I., and readily understood from the diagram, the direction of R is found, and it will be noticed that it makes the same angle, θ, with the direction of Q, as the resultant in Case I., but on the opposite side.

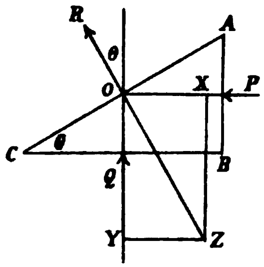


Fig. 82.

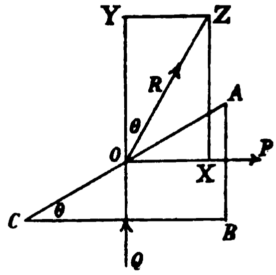


Fig. 83.

Case III., with which we are mainly concerned, is a combination of the conditions obtaining in the preceding instances and may be solved from them. For it is possible to take two subsidiary intensities such that the principal intensity, q, is equal to their sum and the principal intensity, p, to their difference, thus—

$$q = \frac{q+p}{2} + \frac{q-p}{2}$$

$$p = \frac{q+p}{2} - \frac{q-p}{2}.$$

Dealing with these subsidiary intensities in pairs, the problem resolves itself into finding, first, the resultant of two like intensities, each equal to $\frac{q+p}{2}$, as in Case I.; secondly, the resultant of two unlike intensities, each equal to $\frac{q-p}{2}$ as in Case II.; and, lastly, the combined resultant of these two.

In fig. 84, set off $OX = \frac{q+p}{2}$, perpendicular to the plane AC, to represent the resultant intensity due to two like equal intensities of that amount. Next set off $OY = \frac{q-p}{2}$ at an angle $XOY = 2\theta$, to represent the resultant of two unlike equal intensities. Completing the parallelogram, $OZ = r$ will be the resultant of these component intensities in direction and magnitude.

The same result may be demonstrated by a slightly modified diagram, which lends itself to a clearer analysis of the range of stress.

In fig. 85 draw OH at right angles to the plane AC, from the point of intersection O, and set off $OM = \frac{q+p}{2}$. Produce the line of action of the stress Q to L, taking the point L such that OML is an isosceles triangle

with the sides M O and M L equal. With centre M and radius M N $= \frac{q-p}{2}$ describe the arc N_0 N N_1 N_2 cutting M L in N. Join N O, which thus becomes the measure of the resultant intensity r.

The angle θ being variable, the angle H M L $= 2\theta$ will also vary, and with it the angle M O N, which is the obliquity of the direction of the resultant in reference to O M, the normal to the plane, A O. The locus of the point N is the semicircumference N_0 N N_2. The angle M O N attains its maximum value, manifestly, when the direction of r is a tangent to

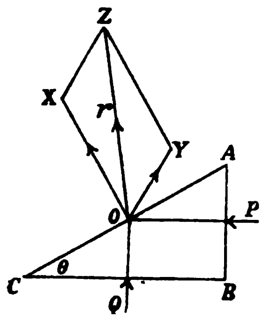


Fig. 84.

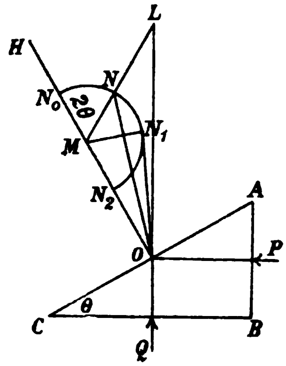


Fig. 85.

the curve—*i.e.*, when the point N coincides with N_1. When this is the case the angle M N O is a right angle, and the angle M O N becomes

$$\sin^{-1}\frac{\text{M N}}{\text{O M}} = \sin^{-1}\frac{q-p}{q+p};$$

Write
$$\sin\varphi = \frac{q-p}{q+p},$$

Whence
$$\frac{p}{q} = \frac{1-\sin\varphi}{1+\sin\varphi}. \qquad \qquad (12)$$

In applying this theory to earth pressure, it is to be noted that the angle M O N represents the limiting angle consistent with equilibrium; in other words, the angle of repose (φ). Equation (12) then determines the minimum intensity, p, of horizontal pressure necessary to maintain the stability of a mass of earth, the measure of whose vertical pressure intensity is q.

In the case of a retaining wall, the earthwork behind which does not rise above a horizontal surface level with the coping, q is equal to the weight of a unit column of earth of height, h—*i.e.*,

$$q = wh.$$

The mean intensity is

$$q_1 = \frac{wh}{2}$$

and the total pressure

$$Q = \frac{wh^2}{2}.$$

Hence, since

$$\frac{P}{Q} = \frac{p}{q}$$

$$P = \frac{wh^2}{2} \cdot \frac{1 - \sin \varphi}{1 + \sin \varphi}, \quad . \quad . \quad . \quad (13)$$

The line of action of P is, as in the case of water pressure, at one-third of the height of the wall above its base.

A simple graphical construction for obtaining the numerical value of $h^2 \frac{1 - \sin \varphi}{1 + \sin \varphi}$ may advantageously be inserted here. Take a vertical line, A B (fig. 86), to represent h, the height of the wall, to any convenient scale, and

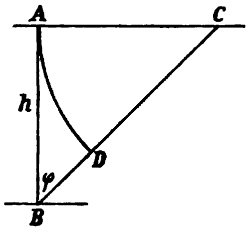


Fig. 86.

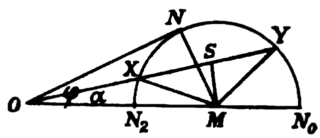


Fig. 87.

from B draw B C, making the angle φ with A B. Draw A C horizontally, and with centre, C, and radius, C A, describe the arc A D. Then B D is the line whose length measures $h \sqrt{\frac{1 - \sin \varphi}{1 + \sin \varphi}}$ to the same scale.

$$\text{For } BD^2 = (BC - CD)^2 = (BC - AC)^2$$

$$= \left(\frac{h}{\cos \varphi} - h \tan \varphi\right)^2$$

$$= h^2 \left(\frac{1 - \sin \varphi}{\cos \varphi}\right)^2$$

$$= h^2 \frac{(1 - \sin \varphi)^2}{1 - \sin^2 \varphi}$$

$$= h^2 \frac{1 - \sin \varphi}{1 + \sin \varphi}.$$

The case of conjugate stresses—viz., that in which the stresses are not mutually perpendicular—is perhaps not strictly essential to the present purpose, as its application is confined to those retaining walls in which the surface of the earth backing is not horizontal—a condition of such rare occurrence in the practice of dock engineering as scarcely to warrant anything in the nature of a lengthy demonstration.* It may be of interest,

* There is only the possibility of a river wall being surcharged by a sloping embankment.

however, to give a succinct description of the method by which the general formula is evolved.

In fig. 87, let the angle N O M $(=\varphi)$ represent the limiting angle of repose, and the semicircle $N_2 N N_0$, the locus of the point N, as in fig. 85.

Through O draw the line O X Y, making the angle M O Y $=\alpha$, the obliquity of the conjugate pressures, and cutting the semicircle in X and Y. Then the limits of the ratio of the intensities of the conjugate pressures are $\frac{OX}{OY}$ and $\frac{OY}{OX}$.

The angle α may have any value between zero and φ. In the former limit, which is the case when the conjugate pressures are perpendicular to each other, and become principal stresses, O X Y coincides with $O N_2 N_0$ and $\frac{O N_2}{O N_0}\left(=\frac{1-\sin\varphi}{1+\sin\varphi}\right)$ is the minimum value of $\frac{p}{q}$. When the obliquity is the greatest possible, such that $\alpha = \varphi$, the points N_2 and N_0 coalesce in N, and the limit of the ratio of the conjugate pressures becomes unity.

For any intermediate position in which α = X O M, the limiting ratio $\left(\frac{p'}{q'}\right)$ of the conjugate pressures may be determined as follows:—Draw S M perpendicular to X Y, and join M X, M Y, each line making the angle θ with X Y.

Then $$\frac{p'}{q'}=\frac{OX}{OY}=\frac{OS-XS}{OS+YS}=\frac{\frac{1}{2}(q+p)\cos\alpha-\frac{1}{2}(q-p)\cos\theta}{\frac{1}{2}(q+p)\cos\alpha+\frac{1}{2}(q-p)\cos\theta}$$

$$=\frac{\frac{q+p}{q-p}\cos\alpha-\cos\theta}{\frac{q+p}{q-p}\cos\alpha+\cos\theta}. \qquad (14)$$

Now, $$\sin\theta=\frac{\frac{1}{2}(q+p)}{\frac{1}{2}(q-p}\sin\alpha,$$

$$\therefore \cos\theta=\sqrt{1-\frac{(q+p)^2}{(q-p)^2}\sin^2\alpha}$$

$$=\sqrt{\frac{(q-p)^2-(q+p)^2\sin^2\alpha}{(q-p)^2}}.$$

And as $$\sin\varphi=\frac{\frac{1}{2}(q-p)}{\frac{1}{2}(q+p)},$$

$$\cos\theta=\sqrt{\frac{(q+p)^2\sin^2\varphi-(q+p)^2\sin^2\alpha}{(q-p)^2}}$$

$$=\frac{q+p}{q-p}\sqrt{\sin^2\varphi-\sin^2\alpha}$$

$$=\frac{q+p}{q-p}\sqrt{\cos^2\alpha-\cos^2\varphi}.$$

Hence, substituting in (14), and cancelling

$$\frac{p'}{q'} = \frac{\cos\alpha - \sqrt{\cos^2\alpha - \cos^2\varphi}}{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\varphi}}. \quad . \quad . \quad . \quad (15)$$

Now, as the stresses are inclined to one another at the angle α, the intensity of the vertical pressures in the case of earthwork will be equal to the weight of a unit column multiplied by $\cos\alpha$.

$$q' = wh\cos\alpha.$$

The mean intensity, therefore, is

$$q'_1 = \frac{wh}{2}\cos\alpha$$

and the total pressure

$$Q = \frac{wh^2}{2}\cos\alpha.$$

Accordingly,

$$P = \frac{wh^2}{2}\cos\alpha\,\frac{\cos\alpha - \sqrt{\cos^2\alpha - \cos^2\varphi}}{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\varphi}}. \quad . \quad . \quad . \quad (16)$$

It will be seen that when the surface of the ground is horizontal $\alpha = 0$, $\cos\alpha = 1$, and

$$P = \frac{wh^2}{2} \cdot \frac{1 - \sin\varphi}{1 + \sin\varphi},$$

as previously demonstrated.

For a surface sloping upwards at the angle of repose, $\alpha = \varphi$ and

$$P = \frac{wh^2}{2}\cos\varphi. \quad . \quad . \quad . \quad . \quad . \quad . \quad (17)$$

According to Professor Rankine, the line of action of the resultant force is always parallel to the surface of the ground. A modification of the theory, due to Dr. Scheffler, determines the direction of the earth thrust as inclined to the horizontal at a constant angle, identical with the angle of repose. In this way, although the total amount of the thrust is greater by Scheffler's hypothesis (being as E G to E F, fig. 88), yet, except in one instance, the overturning effect is less, owing to the nearer approach of the line of thrust to the vertical. The one exception is the case in which the surface of the ground has an inclination φ to the horizontal, and then the two theories lead to the same result.

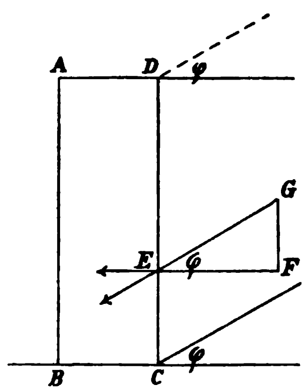


Fig. 88.

Another modification, due to Professor Reilly, takes into consideration the batter, or inclination to the vertical,

of the back of the wall. In fig. 89, the point X is determined by drawing M X at an angle, O M X = 2β.

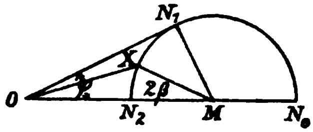


Fig. 89.

Then the total thrust is measured graphically by

$$P = \frac{w h^2}{2} \cdot \frac{O X}{O N_0},$$

or analytically by

$$P = \frac{w h^2}{2} \cdot \frac{\sqrt{1 + \sin^2 \varphi - 2 \sin \varphi \cos 2\beta}}{1 + \sin \varphi}. \quad . \quad . \quad (18)$$

When the back of the wall is vertical, $\beta = 0$, and the equation reduces to

$$P = \frac{w h^2}{2} \cdot \frac{1 - \sin \varphi}{1 + \sin \varphi},$$

which agrees with Rankine's result for similar conditions. The direction of the resultant is constant at an angle γ to the horizontal, such that $\gamma = \beta + \lambda$, the last-named angle being deduced from the equation—

$$\sin \lambda = \frac{\sin \varphi \sin 2\beta}{\sqrt{1 + \sin^2 \varphi - 2 \sin \varphi \cos 2\beta}}. \quad . \quad . \quad . \quad (19)$$

It will be observed that in none of the foregoing expressions is any account taken of the friction exerted by the particles against the back of the wall—a factor which tends to resist displacement. In fact, the assumed conditions only hold good at a suitable distance from the wall beyond the range of its frictional influence.

A formula has been devised by Professor Boussinesq to cover this defect. If ψ be the angle of friction between the wall and the earth, and x the horizontal distance from the face of the wall, the following expressions are given by him for the intensity of horizontal and vertical pressure for values of x less than $\sqrt{\frac{1 - \sin \varphi}{1 + \sin \varphi}}\, h$:—

$$\text{Horizontal pressure} = \frac{w\,(h + x \tan \psi)\,\frac{1 - \sin \varphi}{1 + \sin \varphi}}{1 + \sqrt{\frac{1 - \sin \varphi}{1 + \sin \varphi}} \tan \psi}; \quad . \quad (20)$$

$$\text{Vertical pressure} = \frac{w\,(h + x \tan \psi)}{1 + \sqrt{\frac{1 - \sin \varphi}{1 + \sin \varphi}} \tan \psi}. \quad . \quad . \quad . \quad (21)$$

At the face of the wall x = O, and the expressions become—

$$\text{Horizontal pressure} = \frac{w\,h\,\frac{1 - \sin \varphi}{1 + \sin \varphi}}{1 + \sqrt{\frac{1 - \sin \varphi}{1 + \sin \varphi}} \tan \psi}; \quad . \quad (22)$$

$$\text{Vertical pressure} = \frac{w\,h}{1 + \sqrt{\frac{1 - \sin\varphi}{1 + \sin\varphi}}\tan\psi} \quad . \quad . \quad . \quad (23)$$

Coulomb's Theorem.—What is practically the same formula as that enunciated by Rankine has been developed by MM. Prony and Coulomb, on somewhat different lines, as follows :—

In fig. 90, C E is the line of repose. Were the wedge of earth, D C E, a solid mass it would have no tendency to slide down the plane, C E, the frictional resistance between the two surfaces being sufficient to counteract movement. Evidently, then, if the earth yield at all, it must do so by fracturing along some other plane, the position of which remains to be determined. Meanwhile, assume a position, C F.

Through the centre of gravity of the wedge, D C F, draw K O, vertically, to represent its weight, W. Draw L O, making an angle, φ, with the normal to the plane, C F, to represent the ultimate reaction of the plane, and L K a horizontal line through K. Then the pressure on the back of the wall is measured by

$$P = L\,K = W \tan\theta = \frac{w\,h^2}{2}\tan\theta\cot(\theta + \varphi). \quad . \quad (24)$$

It is now necessary to find the angle which gives the greatest possible value to P. Take the variable factors in the preceding expression, differentiate, and equate to zero.

$$\frac{d\tan\theta\cot(\theta + \varphi)}{d\,\theta} = \sec^2\theta\cot(\theta + \varphi) - \tan\theta\,\mathrm{cosec}^2(\theta + \varphi) = 0.$$

This reduces to

$$\sin(2\,\theta + 2\,\varphi) = \sin 2\,\theta, \quad . \quad . \quad . \quad (25)$$

and, therefore, since the sines of supplementary angles are equal,

$$2\,\theta + 2\,\varphi = \pi - 2\,\theta,$$

$$\therefore 2\,\theta + \varphi = \frac{\pi}{2}$$

$$2\,\theta = \frac{\pi}{2} - \varphi,$$

whence it is evident that the greatest thrust is obtained when the line of rupture, C F, bisects the complement, D C E, of the angle of repose. In this case,

$$P = \frac{w\,h^2}{2} . \tan^2\theta,$$

which is a variant, in form only, of Rankine's expression, since

$$\frac{1 - \sin\varphi}{1 + \sin\varphi} = \tan^2\left(\frac{\pi}{4} - \frac{\varphi}{2}\right).$$

There are, in fact, several different methods of arriving at the same

result. For instance, without using the angle of friction, as in the preceding investigation, take the forces acting at the point, O, in fig. 91, and resolve them along the plane of rupture, C F. Then equate them for equilibrium. The coefficient of friction being $\tan \varphi$, we have

$$P (\sin \theta + \cos \theta \tan \varphi) = W (\cos \theta - \sin \theta \tan \varphi);$$

$$\therefore P = \frac{w h^2}{2} \cdot \frac{1 - \tan \theta \tan \varphi}{1 + \cot \theta \tan \varphi}, \qquad (26)$$

which, when θ and φ are angles such that $\theta = \frac{90° - \varphi}{2}$, is readily transformable into

$$P = \frac{w h^2}{2} \tan^2 \theta,$$

or,

$$P = \frac{w h^2}{2} \cdot \frac{1 - \sin \varphi}{1 + \sin \varphi}.$$

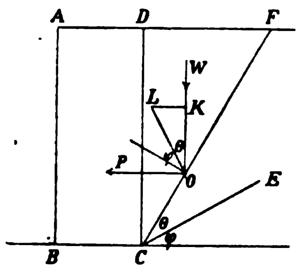


Fig. 90.

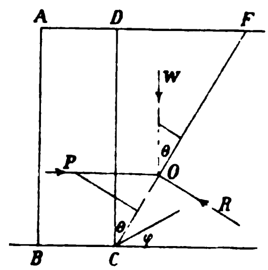


Fig. 91.

*Chaudy's Theorem.**—The undoubtedly excessive values attributed to earth pressure, in the preceding investigations, have led a French engineer to approach the problem from a fresh standpoint, and to evolve a solution which, despite its complexity, yields results more in accordance with practical observation.

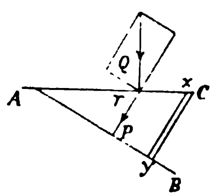


Fig. 92.

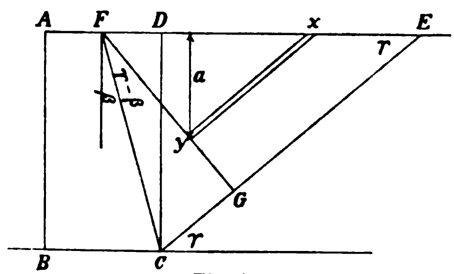


Fig. 93.

M. Chaudy starts with the postulate that a pressure, Q, applied to the surface of a mass of earth causes an oblique thrust, P, and the object of his investigation is to find the amount of this thrust, and the angle at which

* *Mémoires et Comptes Rendus des Travaux de la Société des Ingénieurs Civils de France*, Bulletin de Decembre, 1895.

it exercises its greatest effect. He proceeds to do this by resolving the pressure, Q (fig. 92), into its component parts, $Q \sin \gamma$, and $Q \cos \gamma$, along, and perpendicular to, the direction of the oblique thrust, assumed to make an angle, γ, with the horizontal, and, in this way, he determines the amount of the oblique pressure as

$$P = Q \sin \gamma - Q \cos \gamma \tan \varphi = Q \sin \gamma \left(1 - \frac{\tan \varphi}{\tan \gamma}\right), \quad . \quad (27)$$

the last term being the deduction due to friction.

Considering, now, an element, x, of the surface, A C, as undergoing an intensity of pressure, q, and noting that y, the corresponding element of the surface exposed to the oblique intensity, p, is $x \sin \gamma$, we can derive from the above equation—

$$p\, y = p\, x \sin \gamma = q\, x \sin \gamma \left(1 - \frac{\tan \varphi}{\tan \gamma}\right),$$

whence,

$$p = q \left(1 - \frac{\tan \varphi}{\tan \gamma}\right), \quad . \quad . \quad . \quad (28)$$

which gives the relative intensities of the two pressures.

Applying this to the case of a retaining wall, A B C F (fig. 93), we see that the vertical force for each element of surface is the weight of a strip of earth, $w\, x\, a$, and, therefore, that

$$P = w \times \Sigma\, x\, a \times \sin \gamma \left(1 - \frac{\tan \varphi}{\tan \gamma}\right)$$

$$= \text{area } \overline{\text{F C E}} \times w \sin \gamma \left(1 - \frac{\tan \varphi}{\tan \gamma}\right).$$

Now, the area F C E $= \frac{1}{2}$ F G . C E,

in which $\quad \text{F G} = \text{F C} \cos(\gamma - \beta) = h \sec \beta \cos(\gamma - \beta),$

and $\quad \text{C E} = h \operatorname{cosec} \gamma;$

$$\therefore \text{ the area } \overline{\text{F C E}} = \frac{h^2}{2} \operatorname{cosec} \gamma \sec \beta \cos(\gamma - \beta),$$

and

$$P = \frac{w\, h^2}{2} \,.\, \sec \beta \cos(\gamma - \beta) \left(1 - \frac{\tan \varphi}{\tan \gamma}\right). \quad . \quad (29)$$

When the back of the wall is vertical, $\beta = 0$, and the equation simplifies into

$$P = \frac{w\, h^2}{2} \cos \gamma \left(1 - \frac{\tan \varphi}{\tan \gamma}\right). \quad . \quad . \quad . \quad (30)$$

To determine the value of γ, which will give the maximum value to the equation, differentiate the variable factors, as before, and equate to zero :—

$$\frac{d \cos(\gamma - \beta) \left(1 - \frac{\tan \varphi}{\tan \gamma}\right)}{d \gamma}$$

$$= \frac{\cos(\gamma - \beta)\tan\varphi}{\sin^2\gamma} - \sin(\gamma - \beta)\left(1 - \frac{\tan\varphi}{\tan\gamma}\right) = 0.$$

Multiply by $\dfrac{\tan^2\gamma}{\cos(\gamma - \beta)}$,

$$\therefore \frac{\tan\varphi}{\cos^2\gamma} - \tan^2\gamma\tan(\gamma - \beta) + \tan\varphi\tan\gamma\tan(\gamma - \beta) = 0.$$

Substitute

$$1 + \tan^2\gamma \text{ for } \frac{1}{\cos^2\gamma}, \text{ and } \frac{\tan\gamma - \tan\beta}{1 + \tan\gamma\tan\beta} \text{ for } \tan(\gamma - \beta).$$

Then,

$$\tan^3\gamma - \frac{2\tan\varphi + \tan\beta}{1 - \tan\varphi\tan\beta}\tan^2\gamma = \frac{\tan\varphi}{1 - \tan\varphi\tan\beta}, \quad . \quad (31)$$

a cubic equation which determines the direction of the resultant and its maximum value.

The case of a retaining wall with a horizontal ground surface has alone been dealt with, the investigation of the general case being far too lengthy and involved for insertion. It may be stated, however, that the general formula is deduced as

$$P = \frac{wh^2}{2}\cdot\frac{\cos(\gamma - \beta)}{\cos\beta}\cdot\left(1 - \frac{\tan\varphi}{\tan\gamma}\right)\cdot\frac{\sin\gamma\cos(\beta - \alpha)}{\sin(\gamma - \alpha)\cos\beta}, \quad . \quad (32)$$

and the direction of the resultant is to be derived from the following:—

$$\begin{aligned}\tan^3\gamma &- \frac{2\tan\varphi + \tan\beta}{1 - \tan\varphi\tan\beta + \tan\beta\tan\alpha}\tan^2\gamma \\ &+ \frac{(\tan\varphi + \tan\beta)\tan\alpha}{1 - \tan\varphi\tan\beta + \tan\beta\tan\alpha}\tan\gamma \\ &= \frac{\tan\varphi - \tan\alpha(1 - \tan\varphi\tan\beta)}{1 - \tan\varphi\tan\beta + \tan\beta\tan\alpha}. \quad . \quad . \quad (33)\end{aligned}$$

So much for the purely theoretical aspect of the question which, however, is by no means exhausted. Should the student be desirous of still further investigation, he will find, at the end of the chapter, reference to a few of the sources from which he may obtain additional information.

Weight of Earthwork.—The weight, w, per unit volume of the earthwork behind a retaining wall can only be estimated from experimental results, a number of which are embodied in the following table. Much, however, depends on the degree of humidity of the earth in question, as well as on its actual chemical composition, which, within the limits of the same generic name, may vary considerably. Then it must also be borne in mind that unless the backing consist entirely of carefully selected filling, it is a practical impossibility to accurately gauge for the full extent of the wall the depths of the different strata to be met with. In the majority of cases an estimate has to be founded upon the information derived from a few isolated borings, which may entirely fail to take account of pot-holes or adventitious beds of treacherous material.

TABLE XVI.—APPROXIMATE WEIGHT PER CUBIC FOOT OF VARIOUS KINDS OF EARTH.

	Lbs.
Fine dry sand, loose,	90
,, ,, well shaken,	98
Coarse pit sand,	100
Damp river sand,	118
Quartz sand,	170
Gravel,	90 to 95
Loose, dry shingle,	106
Mud,	102
Dry, common earth, loose,	95
Common earth, slightly moistened,	106
Densest and most compact earth,	125
Loam,	125
Marl,	100 to 120
Clay,	120 to 135
Chalk,	117 to 174
Shale,	162
Rubble filling (with interstices),	100

Surcharge.—The amount of surcharge upon a quay or dock wall can be determined by reference to the weights of cargo to be deposited there and of any superstructure upon the quay. A definite limit, however, is generally fixed in the former case, beyond which wharfingers and others should not be permitted to load quay spaces or shed floors, and an allowance of about 3 tons per superficial yard will generally be found adequate to cover all reasonable contingencies of surcharge. The effect of the surcharge should be considered as extending from the vertical back (actual or virtual) of the wall to the intersection of the line of rupture with the quay surface, and its line of action taken as passing downwards through the centre of this distance. Fig. 94 shows the method of combining the effective pressures due to the earth wedge and the surcharge. The distance, F G, between their respective centres of gravity is divided inversely in the ratio of their weights, and the sum of the latter is taken as acting through the point, K, thus found. It will be noticed that, in this way, the effect of the surcharge is not merely to increase the direct horizontal thrust against the back of the wall, but, at the same time, to raise its point of application and thus still further increase the overturning moment.

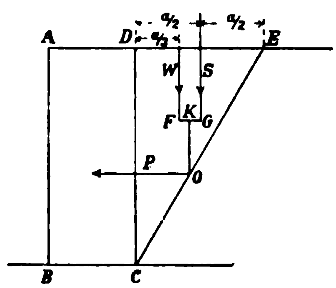


Fig. 94.

Having dealt with those forces which tend to disturb equilibrium, we now turn our attention to those which tend to maintain it.

Restraining Forces.—The magnitude and line of action of the restraining forces are open to less controversy and difference of opinion than is the case with the overturning forces. If the wall have a vertical back the dead

weight of its structure constitutes the one and only element of stability, and its line of action is obviously vertical through the centre of gravity. If, however, the back of the wall be inclined to the vertical at an angle, β, as in fig. 81, the nett weight of the wall must be increased by $\frac{w\,h^2}{2} \tan \beta$, the weight of the earth directly supported by the wall and manifestly assisting to maintain equilibrium. The combined weights must be taken as acting through a common centre of gravity.

Such, at any rate, is the legitimate course to adopt from a purely theoretical point of view. At the same time it must be admitted, on unimpeachable testimony, that the assumption is not borne out by actual experiment. Sir Benjamin Baker states that "he has invariably observed that when a retaining wall moves by settlement or otherwise, it drops away from the filling and cavities are formed. A settlement of but $\frac{1}{32}$ of an inch, after the backing had become thoroughly consolidated, would suffice to relieve the offsets of all vertical pressure from the superimposed earth, and the latter cannot therefore be properly considered as contributing to the moment of stability."* Considering, however, that the purely theoretical aspect of the problem involves equal, if not greater, discrepancies on the other side, in unduly augmenting the effective overturning thrust, it is no inequitable arrangement to regard the advantages accruing to the weight of the superimposed earth as compensating for the neglect of the cohesive power of the backing. Where the offsets at the back of the wall are continued to some depth, it may reasonably be urged that any indisposition of the earthwork to follow settlement in the wall argues a correspondingly high degree of cohesion between the particles and a considerable modification of the calculated thrust.

Another point which calls for attention is the extreme likelihood of water finding a passage beneath the wall, especially in porous foundations, for, in this way, the effective weight of the wall is decreased by the weight of a volume of water equivalent to the immersed section. This may amount to as much as 45 or 50 per cent.; a reduction of great importance. The effect, however, is only felt when the dock is full of water, and then the support derived from the hydrostatic pressure on the face of the wall is sufficient to compensate for the diminution in weight, unless the water in the dock be lowered rapidly while the earth backing is imperfectly drained. The liquid head due to the water imprisoned behind the wall, combined with percolation through the foundation, is sufficient to produce a dangerous complication, resulting in more than one instance, from actual experience, in movement and disruption.

Counterforts, or narrow pilasters, are often built at regular intervals behind a retaining wall with the view of adding to its stability. Their value in this respect is entirely a question of adhesion. In the case of masonry walls it has frequently been found that a separation has taken place between the counterfort and the body of the wall. Such a separation,

* *Min. Proc. Inst. C.E.*, vol. lxv., p. 181.

however minute, is sufficient to nullify the advantages of counterforts, and even to invest them with dangerous potentialities, for, in falling back, they add some portion of their own weight to the earth pressure against the wall. Provided, however, the counterforts be adequately bonded into the body of the wall (and this may be effected very satisfactorily in the case of walls constructed of Portland cement concrete), there can be no doubt as to the advantage to be derived from their aid. The thickness of the wall may then, for theoretical investigation, be regarded as increased to the extent of the thickness of the counterforts, divided by the distance apart at which they are set; in other words, the wall may be taken at its mean thickness. At the same time it is a matter of opinion as to whether the material may not be more economically distributed uniformly.

In instances where it is rendered necessary, additional security may be afforded by the use of *tie-rods* or *tie-bars* firmly connected to the wall near the top and carried to a secure anchorage in the ground some distance away. The very great leverage (measured from the base) at which such a tensile force would act, renders a comparatively slight rod capable of counteracting a considerable degree of earth thrust. The expedient has often been adopted for the purpose of strengthening walls which have showed signs of yielding. Means should be provided for properly tightening up the bars or rods by means of gibs and cotters, screw shackles, or other contrivances. A rough and ready way is to heat the whole length of the bar before completing the attachment; the contraction in cooling will generally be found sufficient to bring the bar into stress.

Weight of Walls.—The weight in air of the various kinds of material of which a dock wall may conceivably be composed is stated below:—

TABLE XVII.—APPROXIMATE WEIGHT PER CUBIC FOOT OF MINERAL SUBSTANCES.

	Lbs.		Lbs.
Basalt,	187	Masonry,	116 to 144
Brick,	115 to 135	Mortar,	109
Brickwork in mortar,	112	Quartz,	165
Felspar,	162	Sandstone—	
Flint,	164	Gatton (Surrey),	103
Granite—		Calverley (Kent),	118
Cornish,	164	Whitby (Yorks.),	126
Aberdeen,	166	Red (Cheshire),	133
Dublin,	170	Craigleith (Edinburgh),	141
Guernsey,	187	Darley Dale (Derby),	148
Limestone—		Talacre (Flint),	150
Bath,	120	York,	157
Portland,	130	Auchray (Dundee),	159
Chalk,	145	Abercarne (Monmouth),	168
Purbeck,	150	Slate—	
Chilmark,	155	Cornwall,	157
Kentish rag,	166	Westmoreland,	173
Marble,	170	Welsh,	180
Magnesian,	175	Trap,	170

Empirical Formulæ.—General Fanshawe's rule was to make the thickness of rectangular revetment walls of brickwork, sustaining ordinary earth, the following percentages of the height:—

For a batter of $\frac{1}{5}$	:	24	per cent.
„ $\frac{1}{6}$	:	25	„
„ $\frac{1}{8}$	:	26	„
„ $\frac{1}{10}$	:	27	„
„ $\frac{1}{12}$	:	28	„
„ $\frac{1}{24}$	:	30	„
For a vertical wall	:	32	„

A rule sometimes adopted for perpendicular retaining walls on railways is to divide the height into three equal parts and make the thicknesses $\frac{1}{3}$, $\frac{1}{4}$, and $\frac{1}{5}$ respectively of the total height.

The following general observations on the subject are given on the authority of Sir Benjamin Baker* :—

"Experience has shown that a wall $\frac{1}{4}$ of the height in thickness and battering 1″ or 2″ per foot on the face possesses sufficient stability when the backing and foundation are both favourable. It has been similarly proved by experience that under no conditions of surcharge or heavy backing is it necessary to make a retaining wall on a solid foundation more than double the above, or $\frac{1}{2}$ of the height in thickness. Within these limits the engineer must vary the strength in accordance with the conditions affecting the particular case." As the result of his own experience Sir Benjamin Baker "makes the thickness of retaining walls in ground of an average character equal to $\frac{1}{3}$ of the height from the top of the footings."

Conditions of Stability.—Having duly selected a provisional sectional profile for a dock wall, and having defined in magnitude and line of action the overturning and restraining forces, it now remains to take the resultant of the latter and consider its effect upon the wall as a whole. The possibilities of failure are threefold—

1. The wall may fail by overturning about the outer edge of its base or of any bed joint. To achieve such a result the overturning moment about these points must exceed the moment due to the restraining force. When the moments are equal there is theoretical equilibrium; but, in order to ensure a sufficient margin of safety, the axis of overturning should be assumed to lie some little distance within the wall—say, at least, $\frac{1}{8}$ of the width of the base.

2. The outer edge of the wall at any horizontal section may be crushed in consequence of excessive compression. This is not likely to arise so much from the actual total weight upon any section as from the unequal distribution of stress. Unless the resultant thrust pass exactly through the centre of gravity of each horizontal plane the stress intensity is not uniform throughout. Uniformity of stress is possible in revetment walls

* *Min. Proc. Inst. C.E.*, vol. lxv., p. 181.

having a considerable backward slope, but from the very nature of their functions this ideal is unattainable in dock walls, and it follows that a certain portion of each bed joint is more highly stressed than the remainder. The intensity is greatest at the outside edge, and, assuming the joint to be a perfect one, it diminishes uniformly as it recedes from the face. If it reach a zero value, it may do so either at the back of the wall or at some point within the wall. The latter alternative should be avoided, as it entails a tensile stress beyond the compressive limit—a stress which bed joints are ill adapted to resist, and which, accordingly, they should not be called upon to undergo. In fig. 95, A B is a horizontal bed joint and R C represents, in line of action and magnitude, the resultant pressure upon the joint. Resolve R into its two components, N R and N C respectively, parallel and perpendicular to A B. The former constitutes a shearing stress, which will be considered later; the latter is the total direct compression upon A B. At A set up the perpendicular $AD = 2\frac{NC}{AB}$. Then, assuming compression to vanish at the point B, join D B and the triangle A D B

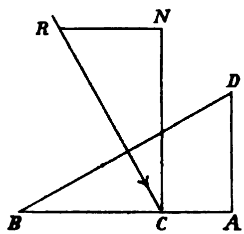


Fig. 95.

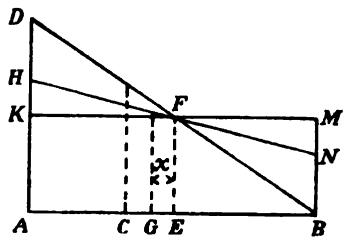


Fig. 96.

will be the graphical representation of the amount and distribution of pressure over the joint, A B. For the area of the triangle $\overline{ADB} = \frac{1}{2}\left(2\frac{NC}{AB} \times AB\right)$ $= NC$. And, since the effect of any system of loading is equivalent to supposing the whole concentrated at its centre of gravity, the line N C necessarily passes through the centre of gravity of the triangle A D B in order to conform to the condition of zero stress at B. Clearly, then, this entails $AC = \frac{AB}{3}$. In other words, the resultant thrust passes through the extremity of the middle third of the wall, but if tension in the joint is to be avoided, it may not exceed this limit.

The resultant passes through the centre of section (E, fig. 96) when there is uniformity of stress throughout, and $AK = \frac{AD}{2}$ is the mean intensity. The stress diagram in this case is, accordingly, a rectangle having the same area. Between the two limits $AE = \frac{AB}{2}$ and $AC = \frac{AB}{3}$ (for we may disregard as inapplicable all values exceeding these) the diagram

will assume some intermediate trapezoidal form. For instance, let G (fig. 96) be the point of application of the thrust: the corresponding stress area will be H A B N. The line H N is defined by the necessity of passing through the point F, and by the following condition:

$$M N = \frac{6 a x}{l},$$

in which $a = $ A K, is the mean intensity of stress, $x = $ G E, is the eccentricity of the thrust, and $l = $ A B, is the length of the base.

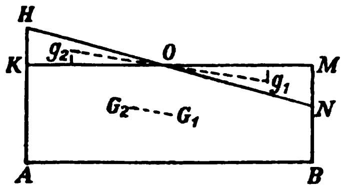


Fig. 97.

The demonstration of this condition depends upon a simple theorem in mechanics.

K A B M (fig. 97) being any body whose weight is W and centre of gravity G_1, if by the transposition of any part of its volume M O N with weight w, its form is altered to the outline H A B N, then the new centre of gravity, G_2, is determined by the proportion

$$\frac{G_2\, G_1}{g_2\, g_1} = \frac{w}{W}, \qquad . \quad . \quad . \quad . \quad . \quad . \quad (34)$$

and the horizontal projections of $G_2\, G_1$ and $g_2\, g_1$ follow the same law.

Now, let us apply this result to the pressure diagram. Call M N y. In fig. 96 H K F and F M N are equal triangles, and the horizontal distance between their respective centres of gravity is, clearly, $\frac{2}{3} l$. Then, in the foregoing equation (34), writing $w = \frac{l y}{4}$ and $W = a\, l$, we have

$$\frac{3 x}{2 l} = \frac{l y}{4 a l}$$

$$y = \frac{6 a x}{l}, \qquad . \quad . \quad . \quad . \quad . \quad . \quad (35)$$

which defines the position of the point N corresponding to any assigned value of x.

A table giving the resistance to compression of various kinds of stone will be found in Chapter iv., and the safe loads on foundations are given on p. 183.

3. The wall may fail by shearing horizontally along some bed joint. The amount of shear is N R (fig. 95), the horizontal component of the resultant thrust. The resistance of masonry joints to actual shearing, which depends largely upon their cohesion, is usually abandoned in favour of their resistance to sliding, which depends on friction alone, and, having a lower value, affords a margin of safety to cover defects in workmanship. In any case this is all the duty which can be expected from the base joint between the wall and its foundation. The amount of resistance to sliding is C N, the vertical component in fig. 95, multiplied by the tangent of

the angle of repose—*i.e.*, of the steepest inclination at which a block of the substance in question will remain stationary. This frictional resistance is quite independent of the area of the surfaces in contact, but its intensity at any point corresponds to the intensity of pressure at the same point. The following are values for the tangent of the angle of repose of several surfaces, usually designated the *coefficient of friction* :—

Dry masonry and brickwork, 0·6 to 0·7.
Masonry and brickwork, with wet mortar, 0·47.
Masonry and brickwork, with slightly damp mortar, 0·74.

Before applying the foregoing principles to a definite example it may be as well to explain one or two methods adopted for finding the centre of gravity of the section of a dock or other retaining wall.

Loci of Centres of Gravity.—The centre of gravity of a square or rectangle lies at the intersection of the diagonals (O, fig. 98).

The centre of gravity of a triangle is at two-thirds of the length of a median measured from the apex (O, fig. 99).

The centre of gravity of a trapezoid is at a point O, fig. 100, on the line A B, bisecting the parallel sides such that $\frac{OA}{AB} = \frac{a + 2b}{3(a + b)}$, where a and b are the lengths of the sides bisected at A and B respectively.

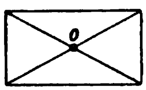


Fig. 98.

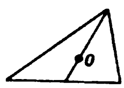


Fig. 99.

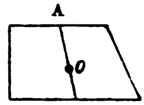


Fig. 100.

The result may be obtained by means of a simple graphical construction. Let A C D B (fig. 101) be a trapezoid. Bisect A B and C D at the points E and F respectively, and join E F. Produce B A to G, so that A G is

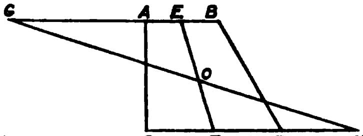


Fig. 101.

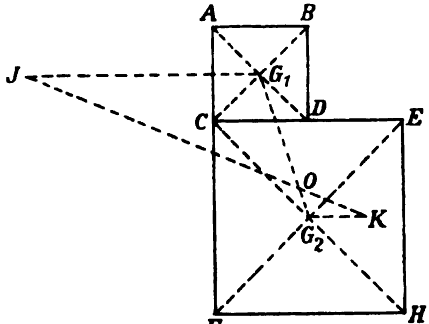


Fig. 102.

equal to C D. Produce C D to H, so that D H is equal to A B. Join G H. The intersection, O, of the lines E F and G H, is the required centre of gravity.

The same principle may be applied to finding the common centre of gravity of two areas. Let A C D B (fig. 102) and C F H E be two areas, whose respective centres of gravity are G_1 and G_2. Join G_1 G_2. From G_1 and G_2 draw two parallel lines, in this case horizontal, but, generally speaking, preferably perpendicular to G_1 G_2, and make G_1 J proportional to the area C F H E, and G_2 K proportional to the area A C D B. Join J K. The intersection of J K and G_1 G_2 at the point, O, gives the common centre of gravity of the two areas.

Sections of dock walls, when not actually forming any simple geometrical figure, may be subdivided into a number of such figures. The combined centre of gravity for the whole figure can then be obtained by the method just described, taking the areas successively and finding the joint centre for each pair. Or any of the following methods may be employed :—

1. In fig. 103 a wall section is shown divided into 3 rectangles. E is the centre of gravity of the topmost rectangle, A C D B, found by inter-

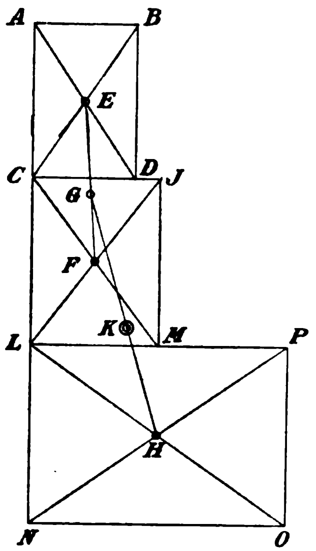


Fig. 103.

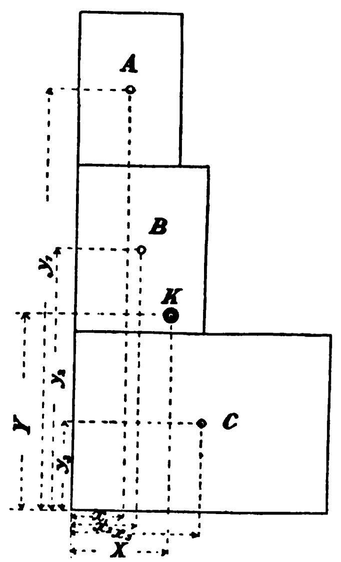


Fig. 104.

secting diagonals. F and H, in like manner, are the centres of gravity for the other two rectangles. Join E F and take a point G such that $\frac{EG}{GF} = \frac{\text{area } CLMJ}{\text{area } ACDB}$. Then G is the common centre of gravity for the two rectangles. Join G H and take a point K such that

$$\frac{GK}{KH} = \frac{\text{area } LNOP}{\text{areas } ACDB + CLMJ}.$$

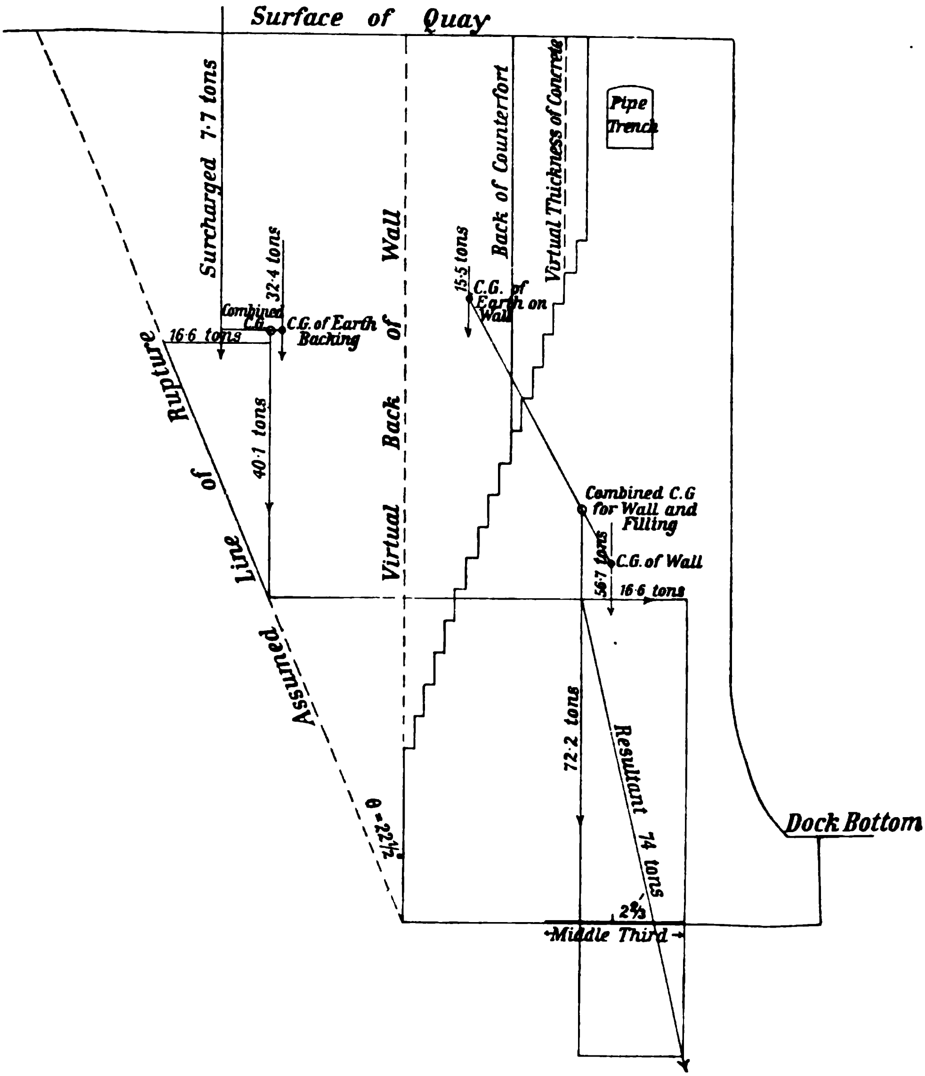
K is the centre of gravity of the whole figure.

2. The point K may be found by combining the co-ordinates of the

subsidiary centres of gravity; thus, in fig. 104, calling the areas of the rectangles A, B, and C respectively—

$$X = \frac{A x_1 + B x_2 + C x_3}{A + B + C};$$

$$Y = \frac{A y_1 + B y_2 + C y_3}{A + B + C}.$$



Scales: 12 feet = 1 inch. 30 tons = 1 inch.

Fig. 105.

3. A very close approximation may be made by the practical expedient of cutting out the profile of the wall in stout cardboard, and suspending it

from two consecutive corners. The intersection of the vertical lines through the points of suspension gives the centre of gravity of the figure. It is better to suspend from three points; the third line acts as a check against possible error.

In all three instances the assumption has been made that the wall is homogeneous. When this is not the case, it will be necessary to deal with the weights of the different sections, instead of their areas. Thus, if two adjoining sections of a wall have areas A and B and unit weights, w_1 and w_2 respectively, their common centre of gravity will be found by dividing the line joining their individual centres inversely in the ratio $\frac{w_1 A}{w_2 B}$.

Typical Example.

Fig. 105 is the profile of an actual dock wall, constructed at Liverpool, to which the methods of stress investigation just described have been applied, with the following results. The material of the wall is Portland cement concrete; the foundation, sound rock; the filling, selected earth and rock rubbish, moistened and well consolidated. The wall is taken at unit length:—

Area of section of wall (deducting pipe trench),	876 sq. ft.
Weight of wall at 145 lbs. per cubic ft.,	56·7 tons.
Area of section of filling resting upon wall,	310 sq. ft.
Weight of filling at 112 lbs. per cubic ft.,	15·5 tons.
Angle of repose assumed at	45°.
Angle of rupture to vertical $\frac{90° - 45°}{2}$,	22° 30′.
Area of hypothetically ruptured wedge,	648 sq. ft.
Weight of wedge at 112 lbs. per cubic ft.,	32·4 tons.
Extent of surcharge,	23·18 lin. ft.
Amount of surcharge at ⅓ ton per sq. ft.,	7·7 tons.
Total vertical thrust at back of wall,	40·1 tons.
Resolved horizontal thrust against wall,	16·6 tons.
Total effective weight of wall and filling,	72·2 tons.
Resultant thrust on foundation,	74 tons.
Overturning moment about outer edge of toe,	340·3 ft.-tons.
Moment of stability about ,,	1101 ft.-tons.
Factor of safety,	3·2.
Width of foundation,	26·5 lin. ft.
Eccentricity of thrust,	2·66 ft.
Average intensity of pressure on foundation per sq. ft.,	2·7 tons.
Maximum intensity,	3·8 tons.
Average intensity of shearing stress per sq. ft. at base,	0·6 ton.
Maximum intensity,	0·8 ton.

Practical Points.

The two essential features of a well-designed dock wall are *weight* and *grip on the foundation.** Without these qualifications even a high moment

* *Proc. Inst. C.E.*, vol. lxv., p. 180.

of stability has proved useless. The stability of a wall depends to as large an extent upon the immobility of its foundation as upon its own inherent resistance to overturning.

The importance of *adequate drainage*, in this connection, has already been alluded to. Where springs or other sources of continuous flow are met with during the building of the wall, they should be conducted to some suitable vent where they may escape freely. Any attempt at repressing them will only result in an outburst elsewhere. Infiltrations of water into the foundation should be dealt with by a temporary drain at the base of the wall leading to a pumping well.

With the same object in view, the *filling* behind a wall for a thickness of 2 feet or so will advisedly be composed of packed rubble stone and broken brick, the interstices of which will act as conduits for subsoil water leading to weep-holes, or outlets, running through the wall at stated intervals. These weep-holes may be formed by drain pipes of from 4 to 9 inches diameter, and they will generally be placed at distances of from 50 to 100 feet, according to the nature of the backing.

Fig. 106.—Old Dock Wall at Leith (1806). Fig. 107.—Quay Wall at Sheerness.

In order to derive as much benefit as possible from the cohesion of the particles, the *earth backing* should be carefully punned in 12-inch layers, and well watered to ensure thorough consolidation.

Offsets in the back of the wall, for the purpose of reducing its thickness, should be narrow and shallow, in preference to being broad and deep, particularly in cases where the foundation is at all unsatisfactory, as the former arrangement is conducive to greater uniformity of pressure.

The *batter* usually assigned to a wall, when the face is not plumb, varies between 1 in 8 and 1 in 24. A battering face to a wall naturally increases its stability, but, at the same time, it detracts from its efficiency. Modern ships have vertical sides with an upper "tumble home," or inward inclination, so that the advisability of, and even the necessity for, walls with plumb

faces become apparent. Old walls are frequently to be found with considerable batter, both straight and curved, as in fig. 106, an old wall at Leith, and fig. 107, a wall at Sheerness, constructed by the late Sir John Rennie. These may be compared with the latest type of quay wall at Liverpool shown in fig. 169.

A curved or splayed *toe* to a wall is a valuable feature, provided it be not carried so high as to nullify the advantage of a vertical face. Prolonged to some distance beyond the face line, the "toe" becomes an *apron*. The former is illustrated in figs. 165 and 169, the latter in fig. 223. The object of an apron is to prevent any abrading or softening action upon the ground in front of the wall, whereby any forward movement would be assisted.

Fig. 108.—Wall at Kidderpur Docks, Calcutta.

Counterforts should be disposed, as far as possible, to form foundations for the bases of columns of sheds, or other structures intended to be built upon the quay. They can be carried up from any offset level. The intervening spaces, instead of being occupied with filling, may in certain cases be arched over, and the vaults thus formed left vacant in order to relieve the pressure. Such arched counterforts are often arranged in two or more tiers. Where circumstances render it desirable to still further lighten a wall, *pockets* may be introduced into its interior, either to be left empty or filled with light material. Fig. 108 shows the Kidderpur Dock wall treated in this way, because of its weak foundation. Walls thus constructed, however,

are very liable to slide forward on their bases, owing to insufficient weight, as actually happened in this instance.

A *trench* or *gallery*, for hydraulic supply pipes and water and gas mains, may often be managed within the body of the wall, at a short depth below the coping level. Access to this will be obtained by manholes placed at convenient distances apart, say, 75 to 100 feet.

In setting out the line of a dock wall, it is by no means desirable to make it absolutely straight, even if intended to be so. Apart from the possibility of some slight forward movement producing an appreciable and unsightly bulge, there is the effect of an optical illusion which causes a perfectly straight coping to appear curved outwards. This latter can be counteracted by giving the wall an almost infinitesimal curvature in the opposite direction. A versed sine of 6 inches in 1,000 feet will generally be found sufficient.

Foundations.—The foundation constitutes so important a feature in connection with the construction of dock walls as to call for some detailed observations. Care should be taken to see that in each case certain essential conditions are fulfilled. These conditions may be stated as follows:—

1. The inclination to the vertical of the resultant pressure upon the surface of the foundation should not exceed the angle of repose of the earth in question. This ensures what is termed stability of friction—*i.e.*, there will be no likelihood of the wall sliding bodily forward upon its base. The condition can always be met by giving a suitable bevel to the surface, so that it slopes downward from the front of the wall to the back.

2. The deviation of the resultant pressure, from the centre of symmetry of the foundation, should not be more than one-sixth of the width. This condition is necessary to maintain absence of tension at the back.

3. The maximum intensity of pressure, at any point, should not exceed a certain limit, dependent upon the nature of the ground. The safe intensity of pressure on natural foundations has been determined as follows:—

On hard rock, . .	9 or 10 tons per sq. ft.
On soft rock and hard clay,	2 to 3 ,, ,,
On sand and gravel, .	1½ ,, 2 ,, ,,
On compact earth, .	1 ,, 1½ ,, ,,
On soft, uncertain ground, .	. ½ ,, ,,

Where an artificial foundation has been prepared, the following intensities should not be exceeded:—

Fig. 109.

For Portland cement concrete, . . .	10 to 12 tons per sq. ft.
For rubble masonry in hydraulic mortar, .	4 ,, 5 ,, ,,

In the case of natural foundations, care must be taken that there is no possibility of lateral escape, and, in the case of artificial foundations, the prepared bed must have sufficient depth to prevent transverse fracture, as indicated in fig. 109. The depth of the bed, *d*, will depend upon the

amount of projection, x. Assuming an ultimate tensile resistance, for good concrete, of 100 lbs. per square inch, and treating the portion x as a cantilever, fracture would occur, with a uniformly distributed load,

$$w = \frac{100\,d^2}{3\,x^2},$$

whence, considering w as the pressure on the foundation, in tons per square foot, and taking a factor of safety of 2,

$$d = \sqrt{w}\,.\,x. \quad . \quad . \quad . \quad . \quad . \qquad (36)$$

4. The texture and chemical composition of the foundation should be such that it is not liable to deterioration from external influences. Certain varieties of rock are softened and washed away by the action of water. The writer has seen sandstone, which required the use of the pick to excavate it, degenerate into the consistency of quicksand after a short exposure to a running stream. Clays are very susceptible to atmospheric influences, expanding and contracting under changes of temperature. Such strata should be covered as rapidly as possible.

5. An unyielding foundation is, *par excellence*, the best, but where this cannot be realised, the foundation must be but slightly and uniformly compressible.

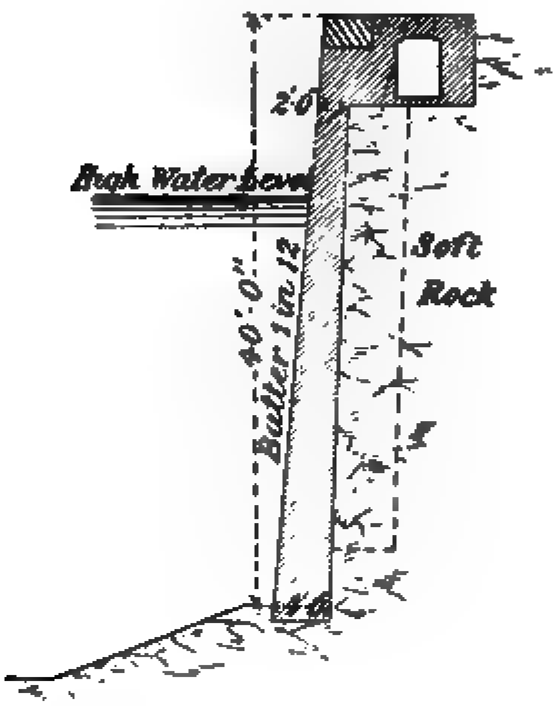


Fig. 110.—Section of Wall. Fig. 111.—Plan.
Herculaneum Dock, Liverpool.

Fig. 112.—Dock Wall at Ardrossan.

The following are a few remarks on prominent varieties of earths :—

Rock, if of a good character, is the most valuable of all bases. It is firm, durable, and unyielding. It involves, perhaps, a little more labour in dressing to a level surface, but, in many cases, inequalities in this direction may be met by benching in steps. Any fissures should be made good, and unsound parts cut away. If the rock be of a soft nature, inclined to pastiness, it should be well drained, and not allowed to remain long exposed. If the site be such that the rock rises very nearly to the surface, the dock

wall may be comparatively economically constructed in the form of a thin veneer of masonry or concrete, securely attached by dovetailing, at intervals, to the vertical face of the rock. Such was the method adopted at the Herculaneum Dock, Liverpool, where the rock cutting was faced by 2 to 4 feet of masonry, with vertical dovetails 5 feet wide and 4 feet deep, at 20-feet intervals, as shown in figs. 110 and 111. If, however, the rock be very hard and durable, the necessity for veneering is obviated, as at Ardrossan (fig. 112).

Clay is a very uncertain material. It varies in volume, texture, and consistency. When thoroughly dry, it is hard and friable; when saturated, it becomes soft and viscous. Mixed with lime, it forms a brittle compound, known as *marl*. When the adulterant is sand, the more tenacious product is called *loam*. Clays possess so many purely local attributes that little can be said of their efficiency, as a class, for foundation purposes, beyond that they are usually satisfactory, if properly protected. One variety of clay—the blue clay—however, possesses striking and dangerous characteristics, which call for especial precautions. Several instances of failures in dock walls have occurred by reason of its treacherous nature. Apparently firm in itself, it often conceals planes of non-adhesion—surfaces in such a state of greasiness that they slide over one another with the greatest facility. These planes may be some distance below the foundation level, and involve the upper stratum of clay in the forward movement of the wall, as actually took place at the S.-W. India Dock.* A blue clay foundation has been responsible for the sliding of dock walls at Southampton, Calcutta, Avonmouth, and elsewhere. Nominally and generally bluish in colour, the upper layers of this clay are sometimes yellow, due to the change of a protoxide of iron into a peroxide, by the action of air and moisture.

Sand and *Gravel* are usually firm and durable foundations, practically incompressible, but they must be confined laterally. They need protection from the action of currents. Very often beds or pockets of these substances are met with in the boulder, drift, or glacial clay. If too deep for excavation, they may be rendered very serviceable by the expedient of mixing some neat cement with the topmost layer.

Stratified Sites.—The question of the depth at which it is desirable to found a quay wall depends not only on the projected level of the dock bottom, but, to a far greater degree, upon the nature and disposition of the strata met with. Having reached a depth adequate from the point of view of design, a problem presents itself which may be resolved into four heads, the first and simplest of which has just been dealt with.

1. A sufficiently firm foundation of indefinite extent. The wall may be erected thereon, with such precautions as the nature of the case requires.

2. A hard stratum overlying a soft one. Here it is essential to preserve the hard covering intact. For example, if a bed of clay overlie a quicksand

* *Min. Proc. Inst. C.E.*, vol. cxxi., p. 120.

it is evident that any perforation of the clay will allow the quicksand to escape under the superimposed pressure.

3. A soft stratum of moderate depth overlying a hard one. In this case it is advisable to found at the lower depth. If actual excavation of the site be impracticable, the desired object may be attained by the use of bearing piles, cylinders, piers, and the like.

4. A soft stratum of considerable depth. Means must be taken to lighten the wall as far as is consistent with its stability, and to distribute the weight over a large area. Framed timber rafts, mats of fascine work, layers of rubble pitching, rows of logs laid horizontally—these are a few of the methods adopted for equalising and reducing the pressure intensity over foundations of this nature.

Artificial Foundations—*Piled Foundations.*—As the use of piles is of wider application than the range of this chapter, they have been dealt with

Fig. 113.—Quay Wall at Rotterdam.

generally in a previous section (Chapter iii.). It only remains to add that, for the purpose of dock walls, a very considerable advantage accrues to the use of *raking* piles. Owing to the obliquity of the resultant pressure, there is a considerable transverse strain upon vertical piles, whereas it is quite feasible to drive the piles at such an inclination that this transverse strain may be avoided, and, with it, the tendency to plough up the ground in front. Instances of piled foundations are shown at Rotterdam (fig. 113), Limerick (fig. 114), Sheerness (fig. 107), and Rouen (fig. 115).

Well Foundations.—The principle of a well foundation consists in causing a hollow shaft or cylinder to sink through a soft stratum by excavating operations carried on from the interior, aided by weighting the circumference, if necessary, until a firm bottom is reached, whereupon the

shaft or cylinder, as the case may be, is filled in solid, and the superstructure erected upon it. The wells are of brick, iron, or concrete, or a combination of any of these. Cylinders being much more common for

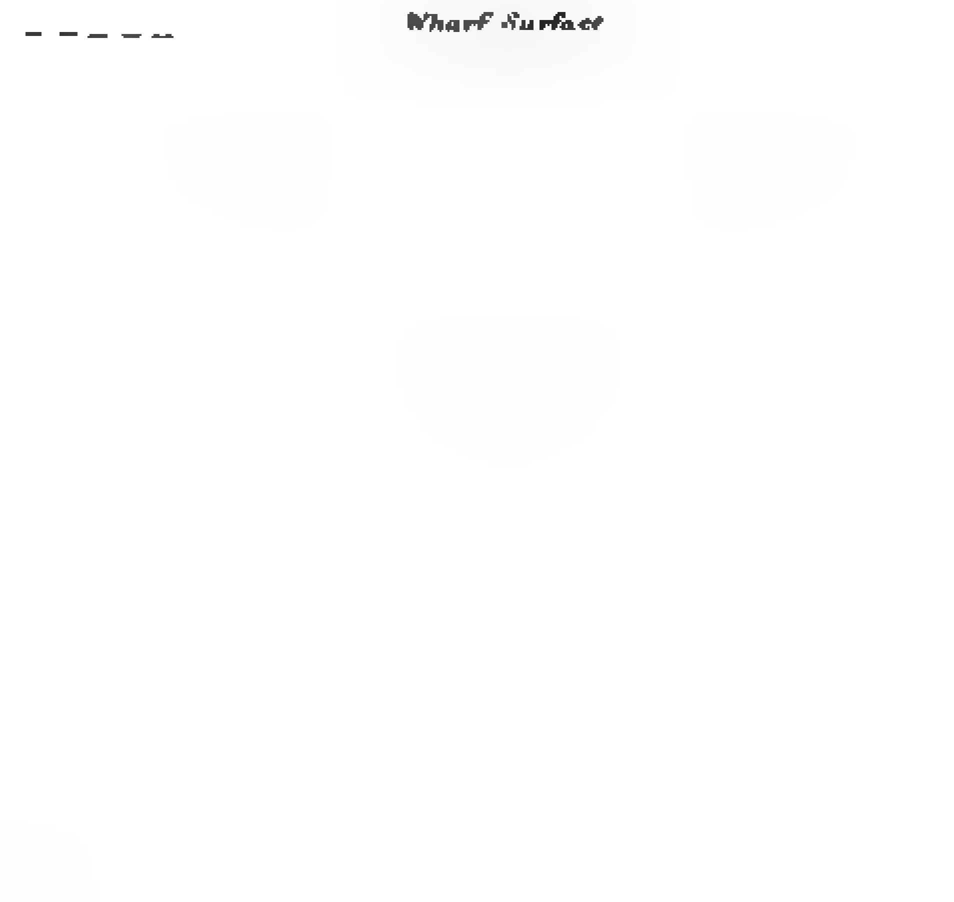


Fig. 114.—Dock Wall at Limerick.

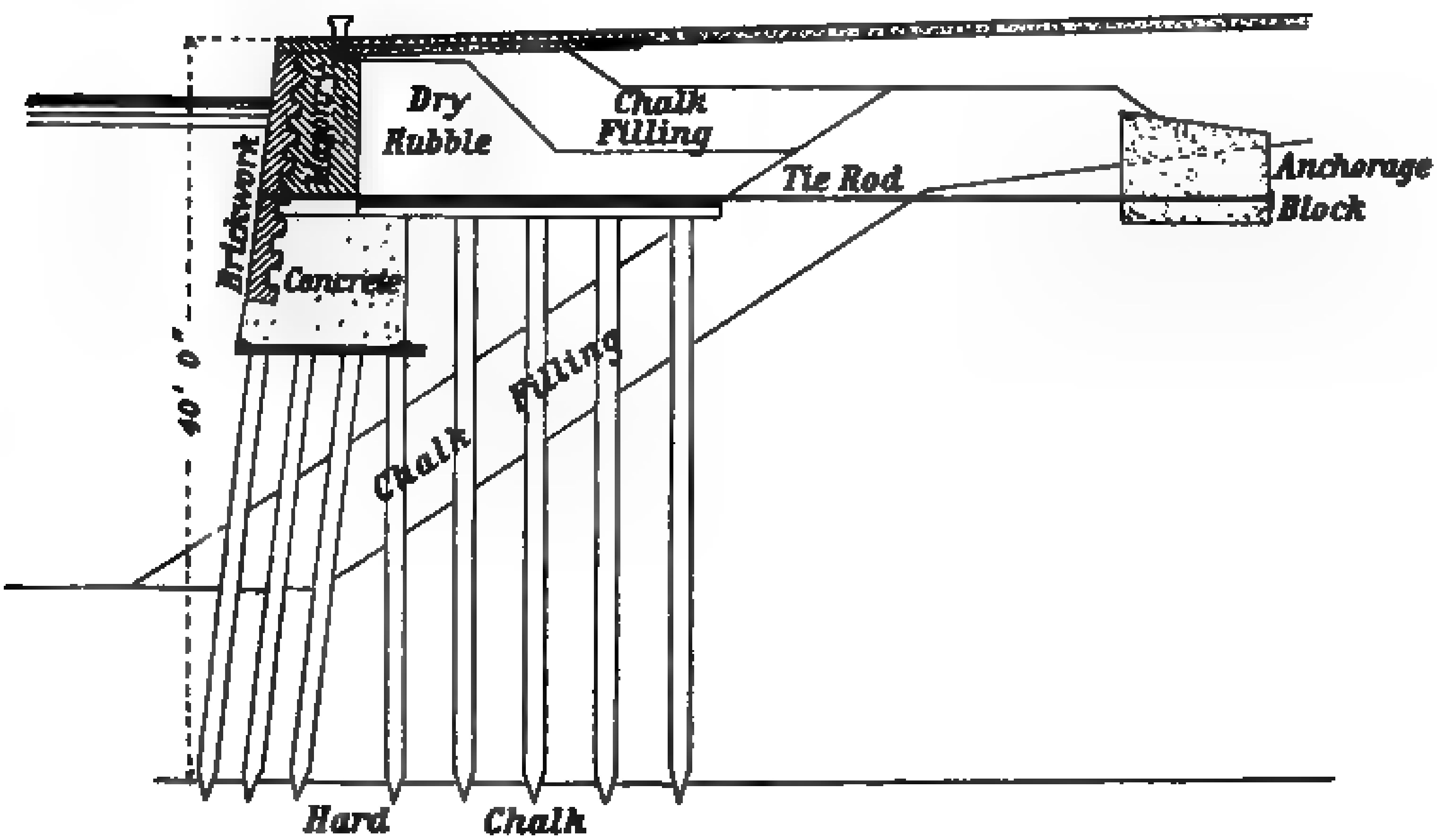
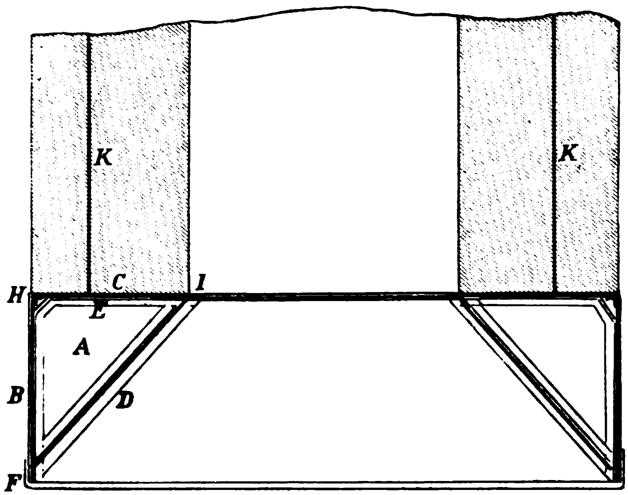


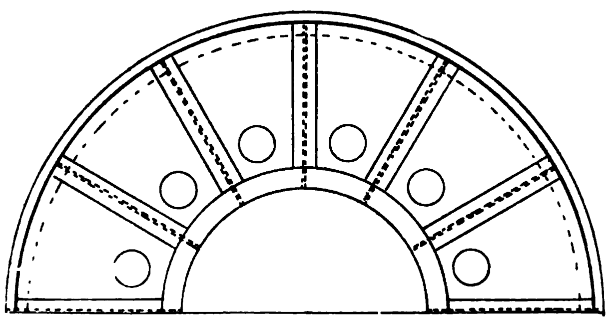
Fig. 115.—Quay Wall at Rouen.

well foundations than rectangular shafts, the former word will be used in the sense of a generic term.

1. *Brick Cylinders.*—In point of antiquity this type of foundation is pre-eminent, having been used from time immemorial for the purpose of well sinking. The method of operations consists in laying upon the surface of the ground a circular curb—formerly of wood, but now universally of



Vertical Section.



Half Plan.

Scale 4 ft. = 1 inch.

Figs. 116 and 117.—Wrought-iron Curb.

metal—in shape like the letter L placed thus Γ, or an angle iron with its uppermost side horizontal. The two wings are strengthened by gusset plates or stiffeners, set at intervals. The curb is not necessarily in one

single piece: for large cylinders, such a base would be inconvenient and impracticable; it is generally composed of segments bolted together. Details of a wrought-iron curb for a bridge foundation in India * are shown in figs. 116 and 117. The height of the curb is 4 feet, and the width of the brickwork base, 3 feet 7 inches. The diameters of the outer and inner edges of the curb are 12 feet 6 inches and 5 feet 4 inches respectively. "The gusset plates, A, twelve in number, framed with angle irons, E, were fixed in position and temporarily bolted to the outside circular plates, from which they radiated inwards, forming in cross-section a V shape; the top of the V being the top segmental plates, C, which were placed upon the gusset frames and fixed with bolts and drifts to the angle-iron ring, H, the whole being riveted together. Finally, the inside sloping plates, D, were fixed and riveted to the angle irons, E, which finished the operation." The spaces between the gussets were filled in with concrete.

Upon curbs similar to the foregoing the brickwork, or *steining*, is founded, vertical bolts (K, fig. 116) being employed to firmly connect the two parts. Excavation, carried on in the interior of the cylinder and beneath its base, causes the cylinder to descend, the action of the cutting edge being assisted by the weight of brickwork above. As the cylinder sinks, brick rings are added continuously until the required depth is obtained.

Great care has to be taken during these operations to maintain the perpendicularity of the cylinder. This, of course, depends upon the equal and uniform settlement of the cutting edge. The most trying time is during the sinking of the first 10 feet or so, and it is recommended that, where possible, the curb should be sunk alone to this depth. The first layer of brickwork may then be some 5 feet in height, and no succeeding layer should be more than 10 feet. It is further recommended that the topmost course of brickwork of each layer should be removed before commencing the next layer, so as to ensure a joint perfectly clean and free from any trace of fallen earth.

Where there is much side friction, the mere intrinsic weight of the cylinder may not be sufficient for the purpose of driving. Additional weight is best added in the form of iron rails and kentledge, which are compact and easily handled. The actual amount of friction to be encountered will depend on local circumstances, but under ordinary conditions it has been found to vary between 3 and 5 tons per square yard. The average rate of sinking in the instance quoted above was 6 feet in eight hours.

Iron Cylinders.—Metal cylinders are almost invariably built of cast or wrought iron, in tiers of tubular castings or of circular plating, the cutting edge being furnished by the lower edge of the bottom tier. Adjacent parts in the case of cast iron are connected by internal flanges, and in the case of wrought iron by fish-plates also arranged internally, with tie and angle-iron stiffeners at intervals. Horizontal flanged joints offer facilities for the

* "Cylinder Foundations" by Imrie Bell and John Milroy, *Min. Proc. Inst. C.E.*, vol. xxviii.

placing of iron kentledge for weighting purposes, and brackets may be specially cast for the same object. This method was adopted in the case of foundations for the piers of a bridge in the River Clyde, the bed of which is running sand to a depth of 80 feet.* Four piles were driven as vertical guides for each cylinder, and uniform subsidence was obtained by systematic distribution of the kentledge. Brackets, 6 inches long, were cast on the lower flange of each length (6 feet 6 inches) of the cylinder, which last had a diameter of 8 feet 4 inches. The kentledge was cast in the form of circular segments, 12 inches thick, so as to fit the concavity of the sides of the cylinder, and rest upon the brackets. In this way 10½ tons dead weight was deposited in five rings upon each tier. Owing to their symmetry and the mutual support afforded by contiguous surfaces, there was no tendency to displacement in any of the pieces. The rate of sinking was 5 feet per working day.

Cast-iron cylinders, 5 feet diameter and 25 feet apart longitudinally, centre to centre, were adopted for the substructure of the earlier quays at Newcastle-on-Tyne (fig. 118).† They were sunk under atmospheric pressure. Over the intervening spaces, masonry and brick arches were turned, springing from cast-iron beams which connected the front and back cylinders. Crescent-shaped rows of metal sheet piling joined the front cylinders below low water level. The superstructure consisted of ashlar facing with concrete backing and granite coping. The wall, however, showed signs of weakness before the dredging in front of it had reached the intended depth, and the work had to be strengthened by a trench of concrete at the back.

Fig. 118.—Quay Wall, Newcastle-on-Tyne.

Elliptically shaped "cylinders" of cast iron in continuous rows were then experimented with, the sheet piling being discarded, but the result was equally unsuccessful. They were found to be too weak to resist lateral pressure. Apparently the failure was due to insufficient thickness of metal, for the substructure of the deep water quays at Cork was satisfactorily carried out in oval-shaped "cylinders" of concrete (figs. 119 and 120).

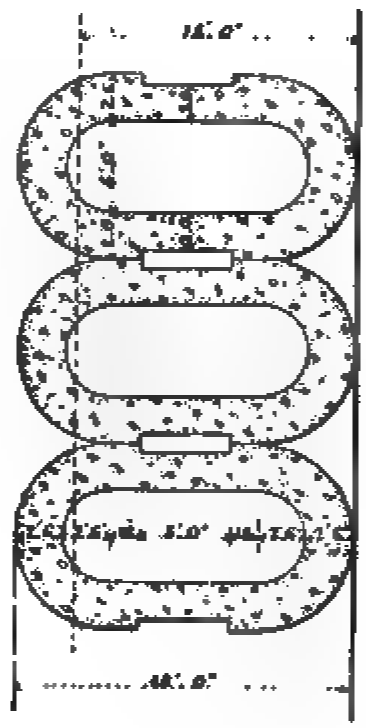
Concrete cylinders present no essential structural difference from those of brick, as already described, their only distinguishing feature being the employment of concrete instead of brickwork for the steining. Perhaps at

* *Min. Proc. Inst. C.E.*, vol. xxviii.

† Scott on "Deep Water Quays, Newcastle-on-Tyne," *Min. Proc. Inst. C.E.*, vol. cxix.

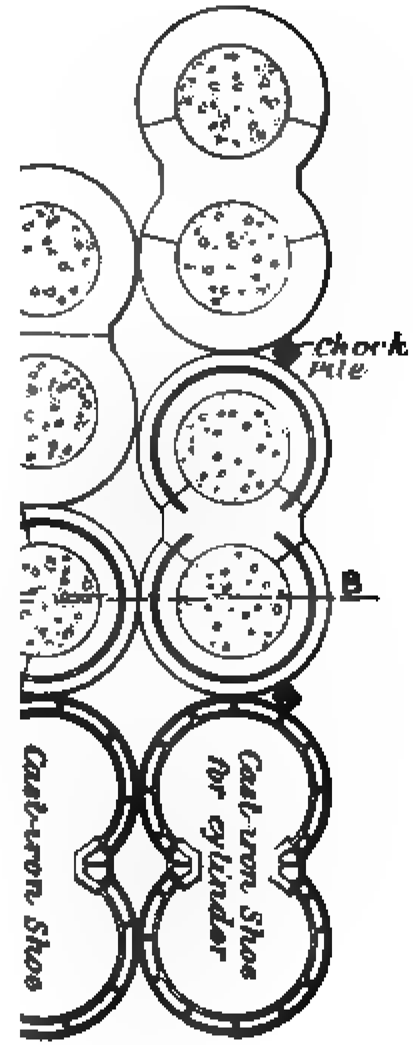
no place have they been practised so extensively or developed to such a degree as in the foundation for the quay walls of the River Clyde. From the elementary series of single cylinders has been developed a dual, and, finally, a triple form shown in figs. 121 and 122, and described in the following extract from a paper on "Clyde Navigation" by the late Mr. James Deas,* the information being revised and supplemented to date by the courtesy of Mr. Archibald Hamilton :—

"The cylinders for carrying the quay walls are triple, 9 feet 7½ inches outside and 5 feet 9½ inches inside diameter. They are made in rings 2 feet 6 inches deep by 1 foot 11 inches thick, in movable wooden moulds on a platform. The concrete consisted of 5 of gravel or broken stones and sharp sand to 1 of Portland cement of the strongest description, mixed together by steam power in mixers designed for the purpose, water being added to bring the mass into a plastic state. To facilitate lifting, the rings were divided into three and four segments, alternately, so as to break bond when built into the cylinders. The division was effected in a simple manner: malleable-iron dividing plates, ⅜ inch thick, were placed radially across the empty wooden moulds in the position required; the concrete was then filled in and well punned with hammers, weighing 25 lbs., so as to secure homogeneity and a smooth surface. Twelve



Figs. 119 and 120.—Quay Wall at Cork.

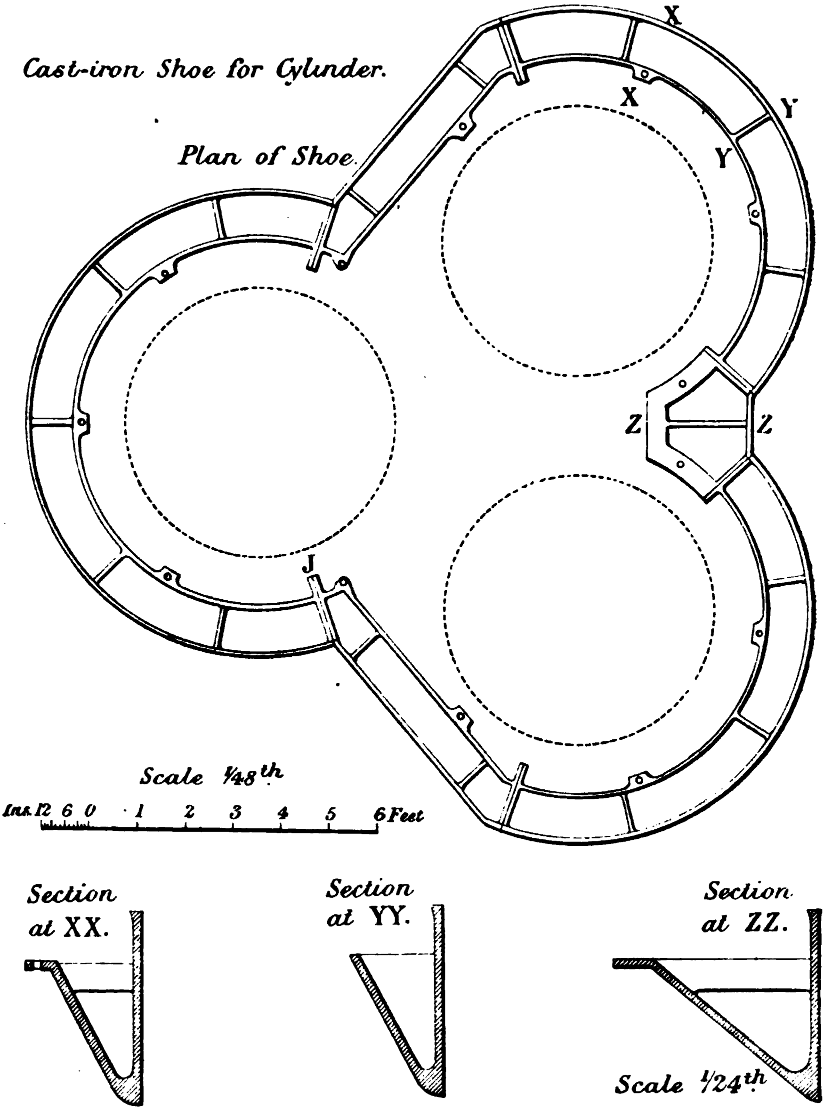
* *Min. Proc. I. Mech. E.*, 1895.



SECTION ON LINE A. B.

Figs. 121 and 122.—Quay Wall at Glasgow.

hours afterwards the dividing plates were withdrawn, and two days later the wooden moulds themselves; and in periods, varying from nine days in hot summer weather to three weeks in the rains of winter, the rings



Figs. 123, 124, 125, and 126.—Cylinder Shoe at Glasgow.

were ready for removal and building. The volume of one ring complete was 10½ cubic yards, and the weight 18 tons, the heaviest segments weighing about 6 tons each.

"The bottom ring, differing from the others, is called a corbelled ring, because it is made less in thickness all round the bottom edge, in order to fit into a cast-iron shoe (figs. 123 to 126), and is tapered inwards and upwards to the full thickness of 1 foot 11 inches. The shoe is of V-shape, 2 feet deep, of 1-inch metal, and the same external size as the rings; the under side of the bottom concrete ring rests on a shelf in the shoe, 6 inches from the top. The wedge-shaped space below is filled with concrete. The shoe weighs about $4\frac{1}{2}$ tons, and is in six parts for convenience of placing in the trench, which was excavated along the line of the quay wall. The bottom of the trench was about 2 feet below low-water level, where it was made 19 feet wide, the sides sloped upwards with a batter of $1\frac{1}{2}$ horizontal to 1 perpendicular. Staging was erected to carry the travelling cranes and digging apparatus. On the bottom of the trench the shoes were placed exactly along the line of the quay wall, and the corbelled ring, being placed on the shelf in the shoe, was bolted to it by thirteen $1\frac{1}{4}$-inch bolts. A malleable-iron washer plate, 5 inches broad by $\frac{1}{2}$ inch thick, was sunk into the top surface of the corbelled ring, in which the recess for this plate and the holes for the bolts passing through the ring had been made in the moulding of the concrete ring. The cylinders, being triple, were placed in the trench so as to dovetail into one another —one in front and two behind, alternating with two in front and one behind. The sides of the groups, where they pressed against each other, were flattened for a breadth of 5 feet so as to ensure a good bearing.

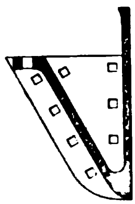


Fig. 126a.—Method of forming Joint at J, fig. 123.

"When the building-up of the rings forming one group of cylinders was completed to the full height, the sand and gravel were dug out, simultaneously, from within each of the three cylinders by means of cranes or excavators specially designed for that purpose. From 400 to 500 tons of cast-iron segmental weights, of the same shape as the rings, were generally required to force each group of cylinders down to the required depth, which is nearly 60 feet below the coping level of the quay. The tops of the cylinders finish about 12 inches above low-water level. The average rate of sinking was about 1 foot per hour; in good working sand as much as 3 feet per hour was attained. When the group had been sunk, each cylinder was cleaned out by means of the excavators to the level of the bottom of the shoe, and was then filled to the top with Portland cement concrete. On this foundation the quay wall is built. In order to effectually close up the apertures between the adjoining groups of cylinders a timber chock pile, 30 feet long by 12 inches square, was driven behind, angleways, so that a sharp corner bears hard against each of the adjoining cylinders.

"The walls are of concrete rubble, and many of the stones weigh from 2 to 3 tons each. The walls are faced with concrete ashlar, in courses ranging from 18 to 15 inches thick; the concrete blocks are not less than

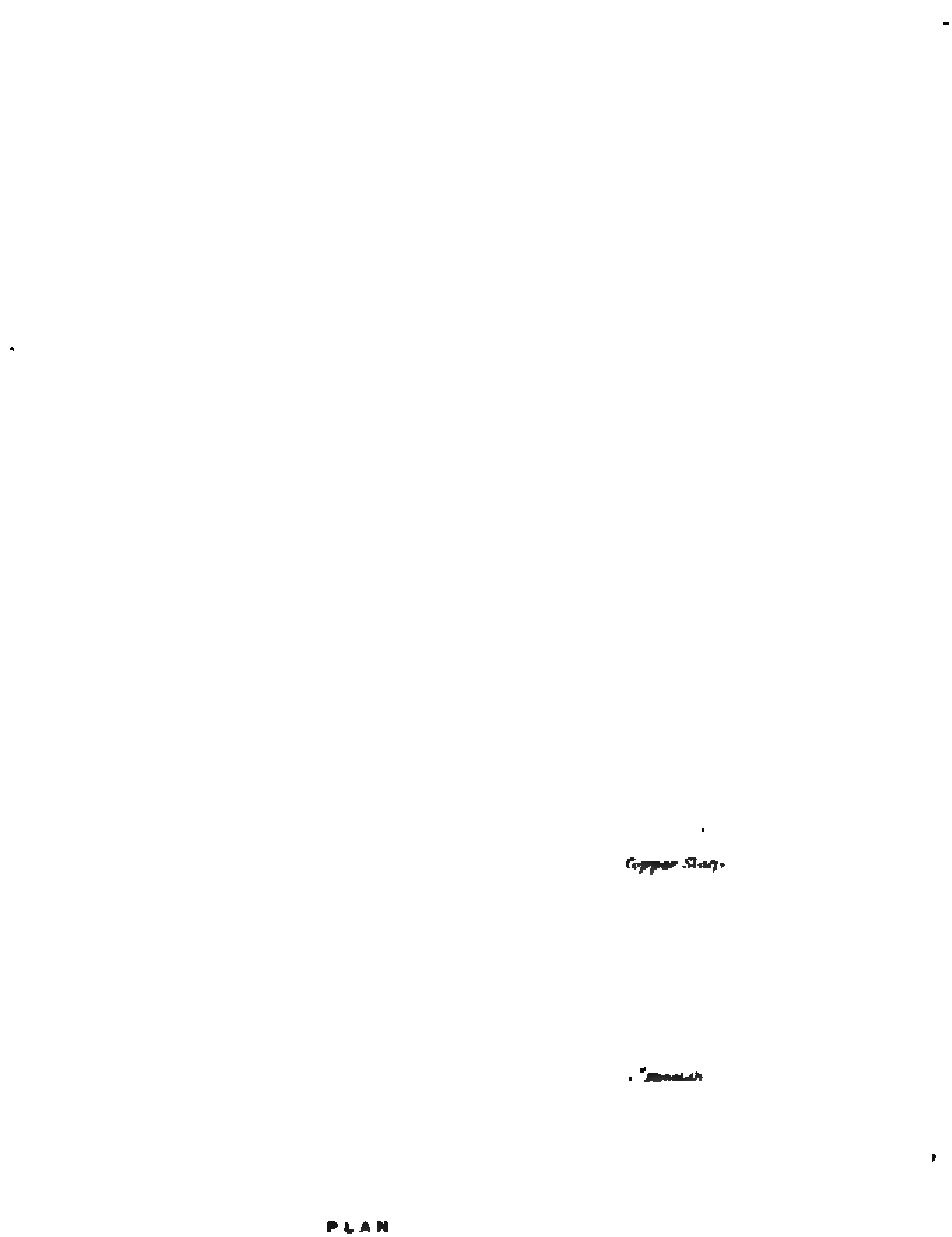
4 feet long by 2 feet broad on the beds, and the headers not more than 10 feet apart from centre to centre. The cope is of granite, $3\frac{1}{2}$ feet broad by 17 inches thick, in lengths of not less than 4 feet.

"To increase the stability of the quay walls of the Prince's Dock, tie-rods, $2\frac{1}{2}$ inches diameter and 60 feet long, were put in, fixed to blocks of concrete masonry, 12 feet long by 6 feet broad and 8 feet deep (fig. 121). Where a depth of 20 feet at low water is afforded, the tie-rods are 64 feet apart, and where there is 25 feet depth at low water, they are 32 feet apart. Where 28 feet depth at low water was desired, the single row of triple cylinders was supplemented behind by a row of twin cylinders, and the tie-rods were increased to $3\frac{1}{2}$ inches diameter and 70 feet length, and placed 64 feet apart.

"Including tie-rods and excavation of trenches, the cost of the walls to give 20 feet depth at low water was £80 per lineal yard; to give 25 feet, £90 per lineal yard; and to give 28 feet depth, £120 per lineal yard."

The same method of construction, with some slight modifications, has been employed in constructing the later quay walls at Newcastle-on-Tyne. The "cylinders" in this case were rectangular in plan, 30 feet long, 20 feet wide, and 37 feet deep, with a rectangular internal cavity 20 by 10 feet, leaving walls 5 feet thick (figs. 127, 128, and 129). The process of sinking was carried out as follows:—

"The curb was 6 feet in height, the cutting edge being an iron casting of V-shaped section, 2 feet 1 inch deep, with vertical wrought-iron straps attached, and timber lining. The cast-iron toe was made in four parts, which were bolted and riveted together at the corners. In constructing the curb, the castings were first set and bolted temporarily together, after which the timber lining (elm or beech) was fitted and bolted upon it. The finished curb was let down into position in four parts, which were bolted together at their corners in the bottom. Sometimes a little concrete was put into the curbs before they were let down. The bottom was levelled to receive the shoes, and was made up, where necessary, to 3 or 4 feet above low-water level. Straps were put across the corners on the inside at the top of the curb to prevent the sides from bulging out. The curbs, being set level, were filled with 5 to 1 concrete, and on this the sides, 5 feet thick, were built all round. The shutters for concreting were 3 feet deep, and were carried on 9-inch by 3-inch standards. After each 3-foot filling sufficient time was allowed for the concrete to set. When the structure had been built to a height of 9 or 12 feet above the top of the curb, it was stripped and sunk, the interior being taken out by grab dredgers until the top was 3 or 4 feet above low-water level." By repeating the process of alternately building and sinking in stages of 9 or 12 feet, the full depth of 37 feet was attained, when the toe of the curb fairly entered into a stratum of hard ballast. "The sinking blocks were not guided or suspended in any way, but were left entirely free and were thus liable to work a little out of place. Sometimes a block would heel over considerably on one side, but could generally be



Figs. 127, 128, and 129.—Newcastle Quay Wall.

righted again by the excavation." Old rails and kentledge were used as sinking weights. The heaviest load was 350 tons.

Having reached a satisfactory depth, a little copper slag was put in the bottom of the wells prior to filling the whole with 7 to 1 concrete containing rubble. Small bags of concrete were packed by divers all round the toe under the curb, and then the bulk of the concrete was lowered in skips through the water to the bottom, and gently released.

The intervening spaces of about 2 feet between adjacent piers were piled, back and front, and concreted.

The superstructure consisted of a sandstone ashlar facing, backed by 5 to 1 cement concrete with granite coping. The face has a batter of 1 in 12.*

General Methods of Construction.

Apart from the means adopted to secure a firm and reliable foundation on sites more or less unsatisfactory and untrustworthy, there are a great variety of methods practised in constructing the dock wall itself; so varied, in fact, as to scarcely admit of any classification, though an attempt will be made here to include some of the more prominent and typical systems under five heads, viz.:—

(α) *Ordinary construction*—
- In the open.
- In trenches.
- Within temporary dams.

(β) *Subaqueous construction*—
- In pneumatic chambers.
- With monoliths.

Construction in the Open.—A description of this method calls for little or no amplification. Where the base rests upon the natural surface of the ground, the wall, if of masonry, is built in the ordinary way, generally with the aid of overhead travellers. If of concrete, it will be necessary to provide means for the support of the face moulds. This may be done by the use of temporary uprights, sometimes called "soldiers." These uprights (fig. 39), placed at convenient distances apart, have a rebate on their inner faces, within which the moulds are free to move vertically. When the latter have been lifted or lowered to their assigned position, they are temporarily fixed by means of wedges. The swivel hooks shown in the fig. are for the purpose of raising the moulds. Alternatively, the moulds may be supported by wooden cantilevers built into the wall at each succeeding course, as shown in fig. 38, and temporarily counterweighted by concrete blocks. These cantilevers can be afterwards cut away to an inch or so within the face line

* Scott on "Deep-water Quays, Newcastle-on-Tyne," *Min. Proc. Inst. C.E.*, vol. cxix.

of the wall, and covered with a thin veneer of cement. Or if their ends be not considered unsightly they may be simply sawn flush with the surface of the concrete.

Where the base of the wall lies below the ground level, the earth may be excavated at any suitable slope until the required depth is reached. If the strata will admit of it, it is preferable to bench out the ground in a series of steps to avoid the formation of a possible plane of rupture between the filling and the natural earth. The steps may even with advantage be sloped downwards away from the wall. Fig. 130 is an illustration of a masonry dock wall built under a combination of the foregoing circumstances. The ground in front of the wall had previously been excavated to the proposed depth; that at the rear of the wall is partially sloped and partially benched. The projection from the back of the wall near the coping level is to form the floor of a trench for hydraulic and other pipes.

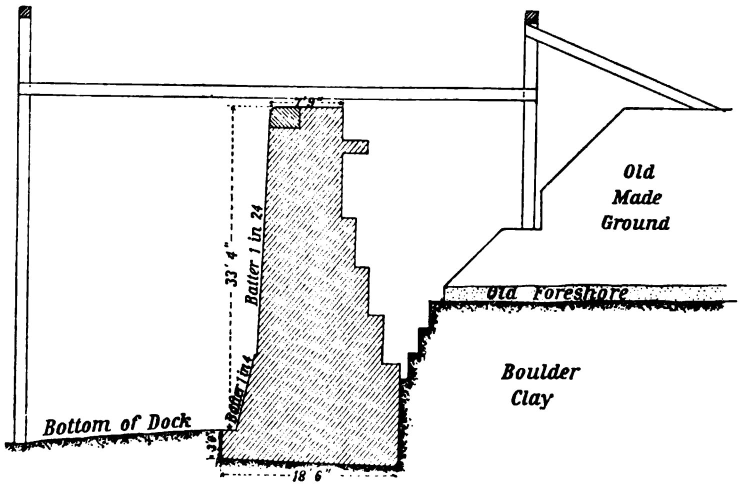


Fig. 130.—Dock Wall at Liverpool.

Construction in Trenches.—The means adopted for obtaining the required depth for the base of a wall by means of timbered trenches is illustrated in fig. 131, which exhibits the actual strata passed through in a definite instance on the banks of the Mersey. The vertical series of shores are placed at intervals of from 10 to 12 feet. The width of the trench at the top is, of course, greater than the assigned foundation width, by the sum of the thicknesses of the timber settings. The small "grip," or trench, in the bottom is for drainage purposes. The method of construction presents no essential difference from those already indicated. The shores and walings which, together with the sheeting piles, are withdrawn as the wall is built, offer facilities for the support of concrete moulds. By this system the earth in front of the wall is excavated at a later stage. In the meantime, any space between the front of the wall and the side of the trench is occupied by

filling tipped in as the wall rises in height. Care must be taken to bring the wall up in regular lifts as far as possible, contemporaneous, and to avoid any extensive "racking back," which causes inequality of pressure on the foundation, and necessitating abrupt changes in the timbering, may induce vertical cracks in the wall.

Trenching was adopted for a quay wall at Belfast as indicated in fig. 132, which also shows the nature of the strata dealt with. Sleetch is the local name for slightly indurated or compact mud.

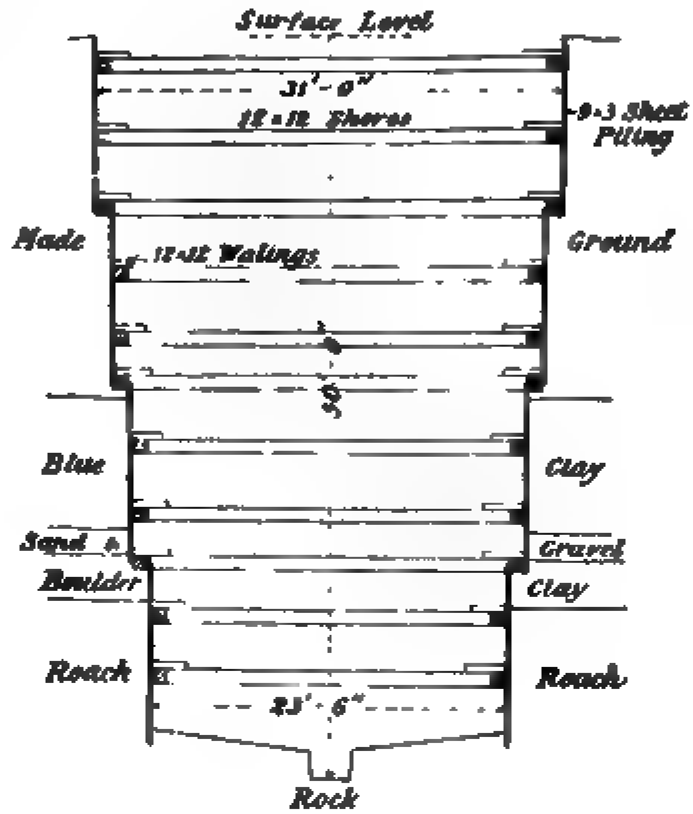


Fig. 131.—Timbered Trench.

Construction within Temporary Dams.—The foregoing sections have dealt with sites more or less inland during the period of construction. Of the many ways in which the work may be carried on when the site is continuously under water, the following is one which admits of constructive work, under normal conditions, after the initial provision of a watertight compartment.

Scale, 20 feet to 1 inch.

Fig. 132.—Quay Wall at Belfast.

Fig. 133 shows a section of a timber dam (a description of it will be found on p. 105, *ante*) which has been floated over the site (at Liverpool), weighted, sunk, and piled. The bottom edge of the dam has been made watertight by means of a tipped bank of clay puddle, which is prevented from slipping away under the softening influence of water by barges sunk on the outer side. Within the enclosed area thus provided, work may

Fig. 133.—Construction within Temporary Dam.

Fig. 134.—Dam and Quay Wall at Ardrossan.

proceed as usual. Pumping power, however, is in this case a more essential feature, not only on account of emptying the dam in the first instance, but also for dealing with leakage, which is sure to be continuous, and the possibility of an inburst of water. Inbursts are most likely to occur in faulty ground, the water being forced, under the great head, through a pervious stratum in the dock bottom. It is, therefore, advisable to dredge the site

clear of all mud and silt before berthing the dam. A good supply of clay will be necessary to replace wastage in the puddle.

The length of the dam in question was 246 feet, divided into 16 bays of 15 feet each, with an overlap at one end. On the conclusion of the work the piles were drawn, and the sides of the dam removed separately. In a similar manner the concrete walls of a tidal basin at Ardrossan were constructed.* Fig 134 is a section of the wall and of the box dam within which it was built.

Construction in Pneumatic Chambers.—This system, in one or other of its forms, represents a very considerable proportion of Continental practice, but it does not seem to have been adopted in any noteworthy instance in English ports, if, as is intended, we restrict the use of the diving bell to the actual construction of the wall. The system dates back some considerable time, and walls have been constructed on its principles, notably at Antwerp, Marseilles, Genoa, and elsewhere.

The following account of its application to the recently constructed quay walls of the Bassin de la Pinède, at Marseilles, is extracted and condensed from an article by M. Batard-Razelière, Engineer-in-Chief of the harbour works there:—†

Fig. 135.—Dock Wall at Marseilles—Section A.

"The foundation of the quay walls is laid on stiff ground (ballast, grit, or hard clay), when that ground is met with above a level of 40 feet below the datum of ordinary low-water level. The profile of the wall is then represented by fig. 135. The masonry is bedded into the ground for a width of about 10 feet at its base. When stiff ground is only to be found below the above-named level, the site is dredged to that depth, the material consisting mainly of mud, sand, and decayed seaweed. A bank of rubble stone is then formed and brought up to a level of 30 feet below datum, having at this level a width of 41 feet, and the normal section of the wall is founded upon this base, as in fig. 136.

To within 5 feet of low-water level the work is executed, by means of compressed air, in the interior of large metallic chambers (caissons ‡), acting like diving bells. From 5 feet below to 18 inches above datum it is

* Robertson on "Ardrossan Harbour Extensions," *Min. Proc. Inst. C.E.*, vol. cxx.

† *Bulletin de la Société Scientifique Industrielle de Marseille*, 2me Trimestre, 1900.

‡ The word "caisson" in this connection has not quite the signification which it has when applied to the apparatus for closing a dock entrance.

executed in the open air, by pumping the enclosed water from the interior of a large, bottomless, metallic chamber, forming a cofferdam. The remainder of the wall, to its full height of nearly 8 feet above datum, is constructed in the ordinary way.

The walls are entirely constructed in ordinary rubble masonry, with the exception of a dressed stone coping and a picked facing down to low water level.

Five caissons are allocated to the execution of the work under compressed air. These caissons are movable, and the work is carried out in such a way as to obtain a continuous block, without any interposition of metal in its interior.

Fig. 136.—Dock Wall at Marseilles—Section B.

Four of these caissons are identical in disposition (figs. 137 and 138). The interior height of the working chamber is 6 feet 6 inches uniformly, but the dimensions in plan vary somewhat. The size of the largest chamber is 66 feet 3 inches by 21 feet 9 inches, the smallest 59 feet by 17 feet 9 inches. Above the working chamber is a compartment having the same horizontal dimensions, in which is deposited the necessary ballast. This ballast is formed partly by a layer of masonry, or of concrete, and partly by iron kentledge. From the roof of the working chamber rise three vertical shafts, situated on the longitudinal axis of the caisson, each surmounted by an air lock above the water level. The middle shaft serves for the workmen; its diameter is, according to circumstances, 2 feet 3 inches or 3 feet 6 inches. The entrance lock is a cylindrical chamber 8 feet 3 inches diameter. The other two shafts serve for the raising of excavations and the lowering of materials; they are 3 feet 6 inches diameter, as also are their locks.

The working chamber is lighted, and the lifts are worked, by electricity

The compressed air is despatched, from a central station on shore, by means of a conduit branching into flexible tubes supported on piles, and is introduced into the top of the central shaft immediately below the floor of the lock. The electric wires follow the same route.

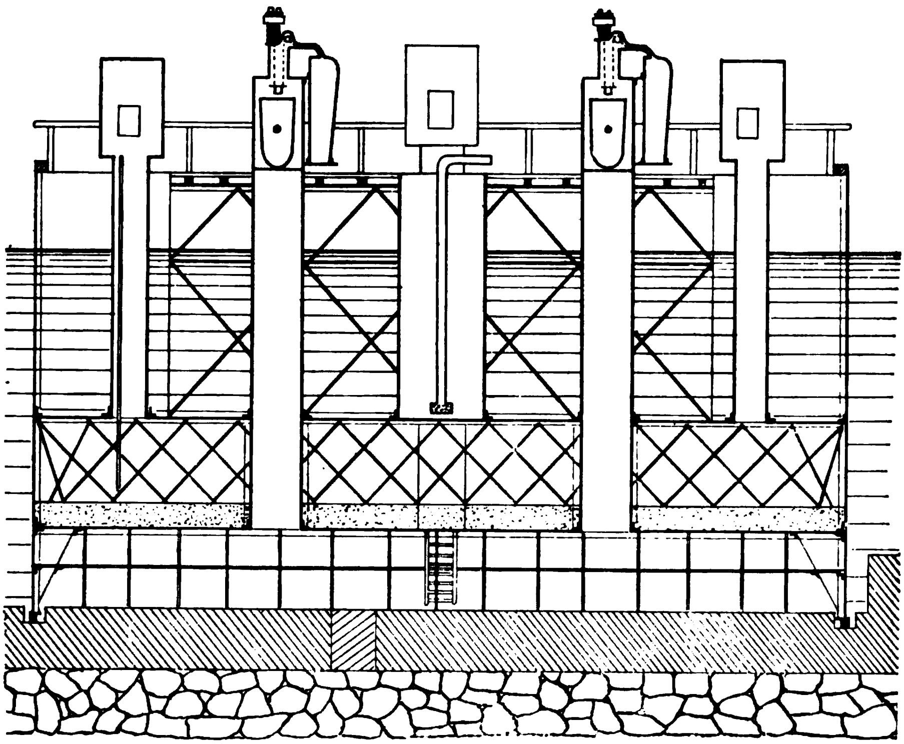


Fig. 137.—Longitudinal Section of Caisson at Marseilles.

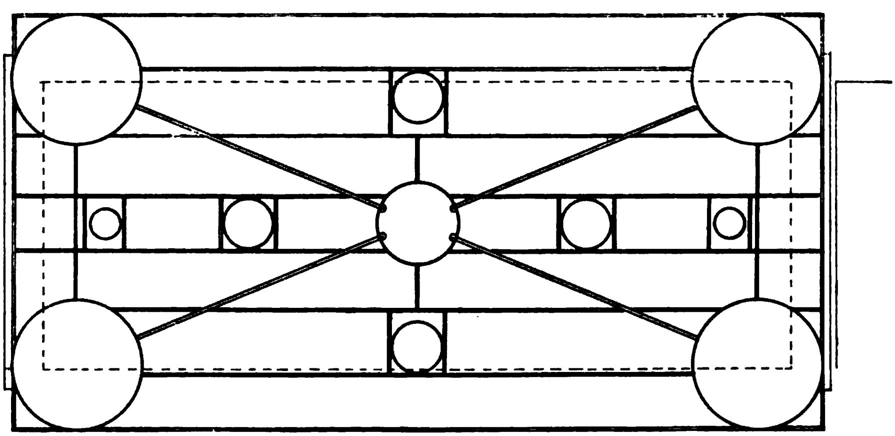


Fig. 138 —Plan of Caisson at Marseilles.

The total weight of a caisson is, on an average, nearly 410 tons, including 350 tons of kentledge. This weight is reduced to about 290 tons when the caisson is immersed, and to 30 tons when it is sunk and the working chamber full of air.

Having dredged and prepared the site, as before described, the caisson is conducted to its place between two barges connected by a framing which forms a deck above the caisson. The latter is then lowered into position and detached from its supports. The working chamber having been filled with compressed air, the surface of the ground uncovered is cleared and levelled, and a block of masonry built upon it about 4 feet in height, its other dimensions corresponding to the size of the chamber and the width of the wall, with a slight clearance in the former case. This completed, the caisson is removed to an adjoining site by a reversal and renewal of the process, the blocks being constructed as closely together as possible and leaving only an interval of about 3 feet between them. A second caisson following the first builds the second course, and at the same time by sitting over the joints between the blocks fills up the vacant spaces with the aid of a diver, who rapidly constructs a brick wall, back and front of the spaces, which are then pumped dry and filled with masonry.

The fifth caisson is self-acting; it can sink or float by its own appliances. It is larger and heavier than the others, and is intended to be worked at variable depths, being used principally for constructing the bottom course of blocks. Its functions generally, however, are the same as those of the other four chambers.

Ordinarily, no excavations were made within the caissons except such as were necessary to prepare a level seat for the wall; in certain cases, however, it was possible to descend about 6 feet below the initial position of the chamber, but there would have been risk in sinking lower, on account of the possibility of not being able to liberate the caisson.

Where firm earth is met with above the floor of the basin, which is the case along and in the neighbourhood of the landward side, the wall is only built to its full width above that level. The lower portion of the wall is simply constructed as a retaining wall or revetment of about 5 feet in thickness, as shown in fig. 139. In this case the caisson is sunk to rest upon the higher level, and the firm stratum below being practically impermeable, the revetment is put in by means of a trench, and the upper portion of the wall is proceeded with as usual.

The rate of working has depended on the nature of the foundation; in the case of very hard ground requiring the pick, the rate of descent of the caisson did not exceed 6 inches per day of 24 hours. In the building of the wall each mason executes about 88 cubic feet of masonry in a shift of eight hours. The construction of one block of masonry absorbed three days, including the manipulation of the caisson and the making good of the joint in the course below. The cost of the masonry, exclusive of the

hydraulic lime, which is supplied to the contractor by the Administration, amounts to about 18 shillings a cubic yard.

The system just described is by no means new, having been practised at the ports of Paimbœuf, St. Malo, La Pallice, and Genoa by the same contractor (M. Conrad Zschokke), but the work now or recently carried out at Marseilles represents its full and perfected development.

With this system may be contrasted the pneumatic process adopted at Antwerp as far back as the year 1877, and still employed for the construction of additional quays within the last few years. The following is an account, necessarily succinct, of the process in its most modern form :—*

The wall (fig. 140) is built of brickwork mainly, with a facing of dressed stone from 2 feet below low-water level up to a coping of ashlar masonry. It rests upon a foundation of concrete of varying thickness, according to the depth of excavation required, but ranging generally between 8 and 16 feet.

Fig. 139.—Dock Wall at Marseilles—Section C.

Fig. 140.—Quay Wall at Antwerp.

The batter of the face is 1 in 10 for the lower portion and 1 in 20 above low-water level. The thickness of the wall at the base is 21 feet 4 inches. The depth of the base is 24 feet 8 inches below low-water level.

A bottomless metallic caisson, rectangular in plan, is floated out over the site of the foundation between two barges, connected by overhead framing. In plan the caisson is 98 feet 4 inches long by 31 feet 2 inches wide. The structure of the caisson will be readily understood from the cross and longitudinal sections shown in figs. 141 and 142. It is lowered into place and sunk to a firm clay foundation by excavating inside of it the alluvial bed of the river. In sinking it is assisted by the weight of the concrete ballast

* *Vide* "Anvers, port de Mer, avec appendice," 1898. Vernon-Harcourt on "Maritime Navigation Works in Belgium," *Min. Proc. Inst. C.E.*, vol. cxxxvi.

immediately above the working chamber, and of the quay wall, which is built up gradually from its roof within an auxiliary cofferdam. The

CROSS SECTION.

LONGITUDINAL SECTION.

Figs. 141 and 142.—Pneumatic Construction at Antwerp.

working chamber is finally filled with concrete through the vertical shafts which have previously served for purposes of access. The interval of about 18 inches, unavoidably left between adjacent lengths of foundation, and the gap of about 42 inches between the sections of wall, are made good by cement concrete, the joint being strengthened by vertical grooves in the connected ends. The wall is continuous above a level of 3 feet above low water. The height of the working chamber is a little over 5 feet, and it projects 8 feet in front of the base of the wall, in order to afford a sufficient area of foundation to support the imposed pressure. The present contractors are Messrs. Hersent & Son, succeeding the original firm of Couvreux & Hersent, who initiated the system.

In contradistinction to the preceding instances, the use of the pneumatic chamber has been applied at the port of Rotterdam to the construction of a portion of the wall considerably above the dock bottom* (see fig. 143).

Fig. 143.—Pneumatic Construction at Rotterdam.

The wall is built upon a timber platform, supported by fir piles driven into the bed of the River Meuse through fascine mattrasses and a layer of sand previously deposited in a dredged trench. The piles are provisionally sawn off at low water, and the caisson, 70 feet long by 29 feet wide, is floated over their heads in such a manner that the ends occupy spaces of 4 feet, specially provided at intervals between the piles, which are otherwise driven at centres of 3 feet 3 inches. The caisson is then sunk until it takes its bearing on the landward side, and at one end upon a portion of the platform already placed in position. At this stage, suitable adjustments of water ballast are made, to maintain equilibrium, and workmen enter the compressed air chamber, which has previously been occupied by water. The

* *Le Port de Rotterdam*, by H. A. van Ysselsteyn, sous-director des travaux de la ville.

piles are cut off to the desired height and fitted with iron collars to support the cross beams, 9 to 12 inches square, which, in turn, carry the flooring, 4 inches thick. Two consecutive lengths of platform are prepared in this way, and then the caisson is berthed over the interval between them, and the decking made continuous. In one week, 14 men working within the pneumatic chamber can completely prepare a length of more than 22 yards of platform.

Upon the foundation thus constructed, the wall is built to its full height with concrete blocks, 8 feet long, 3 feet high, and as wide as the wall is thick, having a facing of basalt. Brickwork has been tried, with unsatisfactory results.

The fascine work is made good between the beams to the underside of the decking, and an additional mattrass is sunk behind the timber work so as to present an upper surface level with the planking. Finally, a mattrass is laid partly upon the platform and partly upon the fascine work behind, and the whole is filled with sand.

A lineal yard of wall constructed in this manner costs at the present day 900 florins, of which one-third may be assigned to the fascine mattrass work. Fig. 113 shows a cross section of the completed wall.

Construction with Monolithic Blocks.—By this system, which consists in building the submerged portion of a quay wall in a series of massive blocks, the use of cofferdams is avoided, and also that of diving bells, except in so far as the latter are found necessary for providing a suitable and level foundation for the blocks. The blocks themselves may be set by means of a floating crane or sheers, and accurately adjusted with the assistance of a diver, who may also, under favourable circumstances, be able to prepare the site for their reception.

Perhaps the most notable instance of the adoption of this method is to be found at Dublin, where the quay walls have a monolithic base course, 27 feet in height, reaching to 3 feet above equinoctial low water. The width of each block at the base is 21 feet 4 inches, forming the entire thickness of the wall; the face length is 12 feet, and the total contents are nearly 5,000 cubic feet of masonry, weighing 350 tons. Adjacent blocks are connected by means of dowels, formed by filling with concrete long vertical grooves, 3 feet square in plan, one-half of which is arranged in the side of each block.

The following particulars relate to the quay wall of a tidal basin built, in 1871,* under the direction of Dr. Stoney, F.R.S., the engineer to the Port Trust.

The necessary preparation and levelling of site were effected by the agency of a diving bell, covering an area of 400 square feet, and furnished with a shaft, 3 feet in diameter, rising from its roof above the surface level of the water, where it was connected to an air-lock for the passage of men

* Stoney on "The Construction of Harbour and Marine Works with Artificial Blocks of Large Size," *Min. Proc. Inst. C.E.*, vol. xxxvii.

and materials. Operations were carried on at a maximum depth of 44 feet. With two gangs of six men, each working alternately in 4-hour shifts, at a cutting 4 feet deep, in stiff clay, the preparation of the foundation for one block occupied about 62 hours.

The masonry of which the blocks were composed consisted of a bulk of irregularly bonded rubble, in pieces not exceeding 2 tons weight, set in cement, with a facing of calp limestone, squared and jointed, the mortar being composed of 4 parts sand to 1 of Portland cement. The blocks were built in wooden frames at a wharf some distance away, and, when ready for depositing, were lifted by a pair of floating sheers. For the purposes of lifting, four wrought-iron suspension bars, 5 inches diameter, having ⊥-shaped extremities, passed through vertical rectangular holes in each block, at the foot of which were circular* cast-iron washers, 2 feet 2 inches diameter (figs. 144 and 145), to distribute the pressure. By turning them through an angle of 90° the bars could be engaged or released.

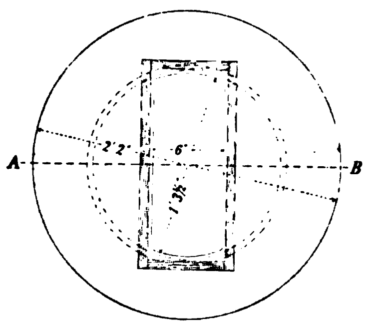
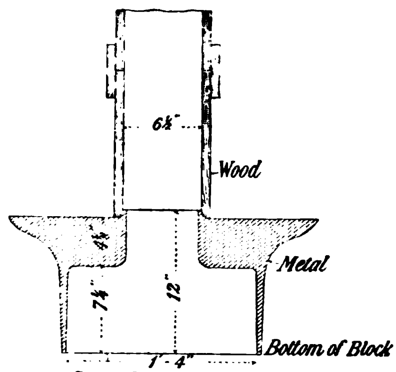


Fig. 144.—Plan of Cast-iron Washer.



Cross Section on Line AB.
Fig. 145.

Conveyance was usually made with the block submerged to half its height, thus relieving the strain on the lifting tackle by some 80 to 100 tons. Arrived at the site, no difficulty was found in bringing the block rapidly into its assigned position. Ranging was performed while it was about 3 or 4 inches off the ground, by means of short timber uprights wedged into the dowel grooves at each side of the block. These were brought into line against a horizontal balk, extending from and attached to the blocks already set. Close contact of adjoining blocks was achieved by the use of a small tackle, the average joint in a length of 300 feet being only ⅜ inch. The dowel grooves were then filled with concrete and the operation was concluded.

The upper portion of the wall was built by tide work to a height of 18 feet 10 inches above equinoctial low water, giving a total height to the

* This, however, is a later improvement; the earliest type of washer was girder-shaped.

wall of 42 feet 10 inches. The coping is of granite in blocks of from 2 to 4 tons weight. The profile of the wall is shown in fig. 146.

The cost of a quay wall constructed in this way and to these dimensions came to £40 per foot run, including 7 per cent. interest on a sum of £33,800 for plant. The rate of construction was 400 lineal feet per annum.

On the same principle, but with blocks of smaller dimensions, a quay wall (fig. 147) some 500 yards in length, was constructed at Cork about the year 1877.* The submerged portion of the wall consisted of three rows of blocks, rectangular in plan, weighing from 35 to 49 tons each. As in the case of the Dublin blocks, they were constructed at a wharf some distance away and transported to their respective positions by a floating sheers. The composition of the monoliths, however, was different, in that they were made entirely of concrete in the proportion of 7 to 1—viz., 5 parts river ballast, 2 parts broken limestone passed through a 3-inch ring, and 1 part Portland cement.

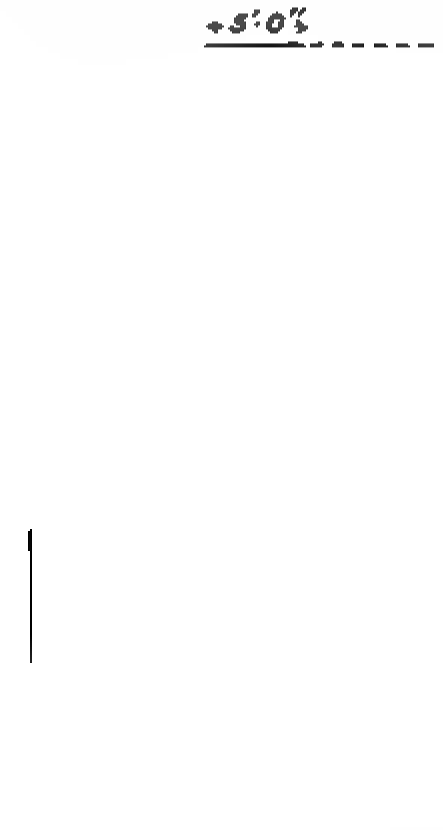


Fig. 146.—Quay Wall at Dublin.

Fig. 147.—Quay Wall at Cork.

The foundation consisted mainly of fine compact gravel and sand. After being dredged to within 2 feet of the required depth the remaining material was removed by divers. A rectangular frame of angle iron slightly larger than the block was then laid on the ground and adjusted by soundings from above. The surface inequalities within the enclosure were levelled by an iron straight edge.

The blocks (figs. 148 and 149) were suspended by four stirrup-rods passing down vertical grooves, 10 inches by 5 inches, in the sides of the blocks,

* Barry on "Deep Water Quays at the Port of Cork," *Min. Proc. Inst. C.E.*, vol. c.

which were afterwards used for the reception of 10-inch square stone dowels, 3 to 4 feet long, to connect adjoining lengths. The ends of two small wrought-iron girders in recesses, at or near the bottom of each block, rested in the stirrup-rods, and all were withdrawn together at the close of the setting operations.

The superstructure consisted of a facing of regularly coursed limestone ashlar, backed by 6 to 1 concrete, with a coping of Cornish granite.

Another instance of monolithic construction, with yet smaller blocks of concrete, is to be found at Kurrachee (fig. 150). The dimensions of the blocks were 12 feet by 8 feet by 4½ feet, and their weight 27 tons each. Lifting and setting were performed entirely by land carriage with the aid of a Titan, which travelled over the sections of work already executed and deposited the blocks in front of it. The depth of the foundation bed was 15 feet below the surface level of the water, and the blocks were laid in three horizontal tiers or courses to a total height of 24 feet 6 inches. The blocks were not set vertically, but with a slight backward inclination as shown in fig. 150. The sea bottom was sandy at a depth of 25 to 30 feet, and was surmounted with a rubble foundation, levelled by divers, and upon which the blocks were laid.

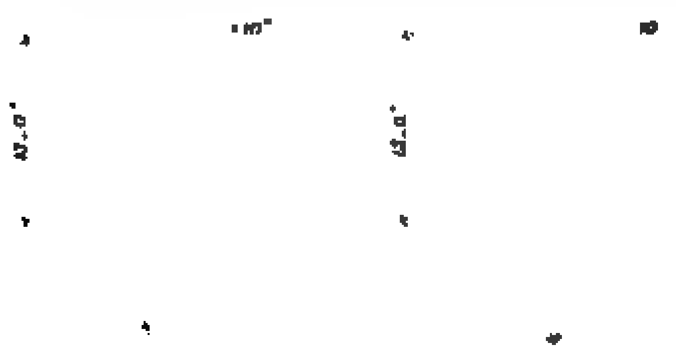


Fig. 148. Fig. 149. Fig. 150.—Blockwork at Kurrachee.

A similar method was adopted for building the quay walls at Suez. The blocks, which were about the same size as those at Kurrachee, were conveyed to their destination in barges.

Other examples may be quoted from ports in the Mediterranean, at Marseilles and elsewhere. The French were, in fact, the pioneers of the system, when they inaugurated it at Algiers as far back as the year 1840. It is still being practised for harbour work in Algeria at the present time, and the following particulars, furnished by the courtesy of the Engineer in charge, M. Georges Boisnier, relate to a quay wall at the port of Bougie, now under construction (see fig. 151).

The sea bottom is mud to a considerable depth, and in order to obtain a sufficiently broad area for the pressure, a foundation of rubble stone, 11½ feet in depth, is deposited within a trench dredged to a bottom width of 55 feet. The wall consists of five tiers of masonry blocks of varying size, only one of which is above the surface of the water. The blocks are constructed on a neighbouring quay with limestone from a local quarry. Those in the

two lowermost tiers weigh about 35 tons each, the upper tiers average 5 tons less. An interval of from three to four months is allowed to elapse between making and using, when the mortar is composed of hydraulic lime, but only three weeks, when of cement. The blocks are set by a floating crane with the assistance of a diver. When the four submerged courses have been constructed, the wall is weighted with a temporary surcharge of two tiers of blocks, which causes the structure to settle bodily to the extent of about 3½ feet in a period of two months, at the end of which time the rate of settlement is found to be insignificant, the surcharge is removed and a coping course substituted. The backing behind the wall is of rubble with a covering layer, 3 feet thick, of quarry spalls, above which is discharged the mud dredged from the foundations.

Fig. 151.—Section of Quay Wall at Bougie.

The cost of this type of wall works out to rather more than £14 per foot run, made up, approximately, as follows :—

Dredging site, .	£1	1	0
Rubble filling, .	4	15	0
Artificial blocks,	6	3	0
Surcharge, .	0	9	0
Coping, . .	0	11	0
General, . .	1	5	0

Experience has shown inadequate stability in a portion of the wall, as constructed above, and several important modifications are being introduced into another section of the same undertaking. The dredged mud is no

longer used for any part of the backing, its place being taken by dry quarry rubbish. The blocks are made to larger dimensions, but, in order to facilitate setting operations, they are rendered temporarily lighter than they would otherwise be by the arrangement of voids or pockets in their interiors, as shown by the plan in fig. 152. The lowermost blocks weigh some 50 tons prior to the filling of the pockets with concrete, an operation which is performed when they are in position. The former face batter of 1 in 10, found to be unsuitable for vessels with vertical sides, is now reduced to 1 in 20.

The profile thus adopted may be compared with that of a quay wall at the neighbouring port of Sfax* in Tunis, similarly constructed, but with the face receding in a series of offsets as shown in fig. 153.

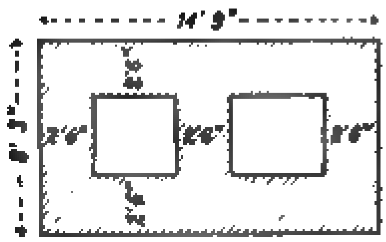


Fig. 152.

Fig. 153.—Quay Wall at Sfax.

The difficulty caused by excessive settlement in walls of this class is well illustrated by the case of a wall at Smyrna, where no less than six or seven tiers of blocks had to be superimposed, instead of four, as originally intended, while the front of the wall had to be supported by a rubble mound carried up to within 7 feet of mean sea-level.

Failures.

Failures of dock walls are by no means scarce, and they often present interesting and instructive features, but, in nearly every case, the cause can be traced to a bad foundation. Movement to a greater or less degree is to be expected, and has been experienced in all walls founded upon any other stratum than hard rock. It is stated as the experience of Voisin Bey, the Engineer-in-Chief of the Suez Canal, that he had never found a long line of quay wall which, on close inspection, proved to be perfectly straight in line and free from indications of movement.

* Baron de Rochemont on "Quelques Ports de la Mediterranée," *Int. Nav. Cong.*, Paris, 1900.

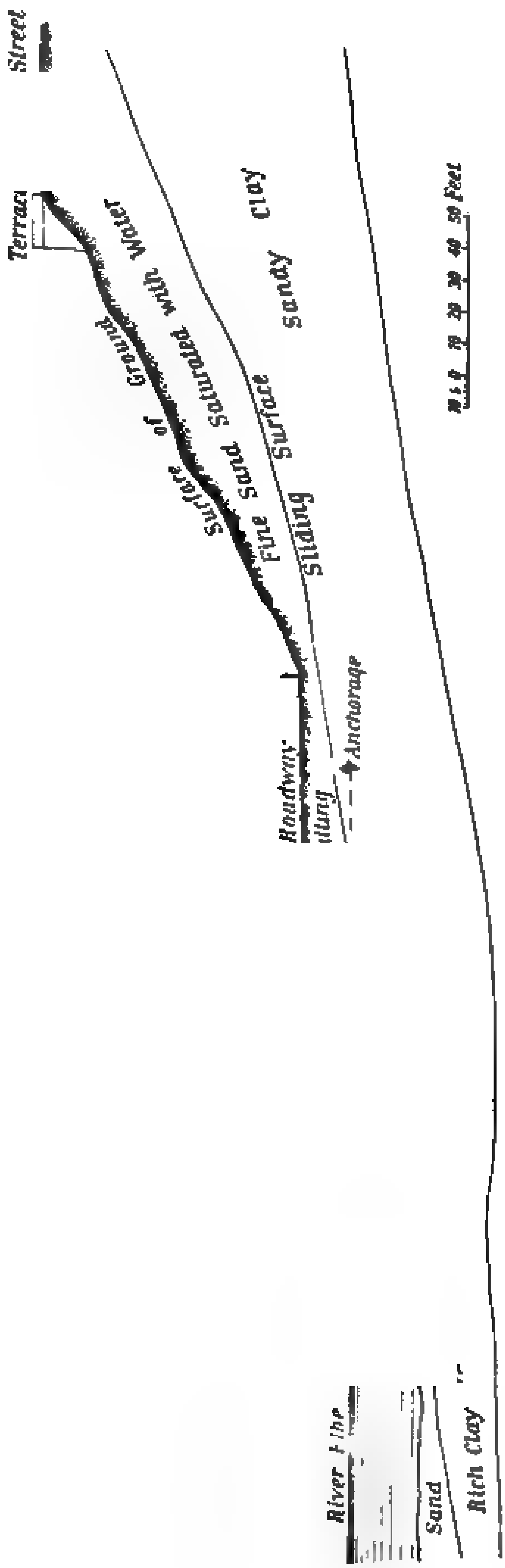


Fig. 154.—Quay at Altona.

As has already been pointed out, the most treacherous of all strata, from the point of foundation for a quay wall, is the blue clay. Out of many instances, which might be cited as evidence of its dangerous nature, the following account of the sliding forward of a wall at the port of Altona is selected as affording an interesting example of remedial measures adopted with perfect success:—*

The town of Altona is situated on the right bank of the River Elbe, and the level of the ground rises gradually from the river bank inland to a height of 105 feet. The town stands partly upon this slope and partly upon its summit.

The uppermost stratum of the site (fig. 154) consists of very fine sand, interspersed with numerous water streaks. Below this sand is a layer of clay, which rises to the hills at an angle rather less than the surface inclination. The clay is firm when not saturated with moisture. It is, however, soluble in water, and becomes a smooth, soapy body, offering no effective resistance to slid-

* *Min. Proc. Am. Soc. C.E.*, vol. xxx.

ing. As long as the water from the hills can percolate freely through the sand and escape there is no danger, but when the outlet is blocked the sand becomes sodden, and the clay acquires a slippery surface conducive to landslip.

The quay wall consists of a solid face of three thicknesses, 4½, 6, and 9 feet respectively, formed by offsets at 5½ and 9½ feet above the base. It is backed by a series of counterforts, arranged at intervals of about 30 feet, and well bonded into the wall. The spaces between the counterforts are spanned by two tiers of arches, the lower of which sustains the sand filling behind the wall, and the upper forms a foundation for the line of steam cranes which serve the quay front. The materials of which the wall is constructed are hard bricks and cement mortar, the latter in the proportion of 1 of cement to 4 of sand.

The wall rests upon a level base, formed by a strong horizontal mortised framing of longitudinal and transverse timbers, covered with planking and supported by vertical and oblique bearing piles. A row of tongued and grooved sheet piling is driven to retain the bank of earth below the platform level. A corrugated iron shed, founded upon a distinct system of piling, stands a little distance back from the face of the quay.

In August, 1890, shortly after the completion of the work, cracks were observed in the wall and in the brick gables of the shed, and it was found that both the quay and the shed had perceptibly shifted their positions. The backing at the rear of the wall was removed forthwith, in order to lighten the pressure. Very shortly afterwards the movement of the shed was found to have been arrested, evidently by the resistance of its foundation piles on the landward side, which had been driven well into the lower clay.

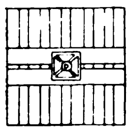
Meanwhile, the quay wall continued a slow but uniform movement outwards; so gradual and minute, however, as to permit a series of observations to be taken systematically, from which the source of the mischief was accurately traced, and the means devised for remedying it. It was found that the whole stratum of earth above the clay, extending as far as the hill top, was sliding bodily forward towards the Elbe, and, as the lower ends of the quay piles were bedded in the clay, the upper masonry was turning about the feet of the piles as about a pivot.

In order to check this movement, and restore the stability of the wall, a series of 29 iron stays, placed about 15 feet apart, were secured to the upper face of the quay piles, and led to anchorages, some 164 feet back, sunk well below the surface. These stays consisted of links about 16 feet long and 10 square inches sectional area, alternately of round and bar iron, the latter being double and attached to the rods by bolts passing through their ends. At the quay face the extremity of the stay was made fast to a heavy iron plate, bearing against horizontal beams below the surface of the water, which transmitted the pressure evenly to the foundation piles. The anchorage on the landward side (figs. 155 and 156)

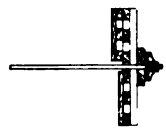
consisted of a stout shield of nearly 70 feet surface, made of strong logs, abutting against a smooth vertical face in the clay.

As a further precaution, the soil behind the quay was excavated to below low-water level, and the void filled with broken brick, which gave a backing of a lighter character, while, at the same time, it resulted in more efficient drainage. The arches between the counterforts having been destroyed by unequal settlement, a light concrete wall was formed behind them, to take the surface pressure and transmit it to the bearing piles at the base of the wall. The quay line of rails is now carried on iron joists 6 feet apart, bedded in concrete, and spanning the space intervening between the two walls.

The work took eighteen months to carry out, cost about £30 per lineal foot, and has proved satisfactory, in every way, since the quays were re-opened to full traffic in 1892.



Elevation of Anchorage.



Vertical Section of Anchorage.

0 1 2 3 4 5 10 Feet

Figs. 155 and 156.

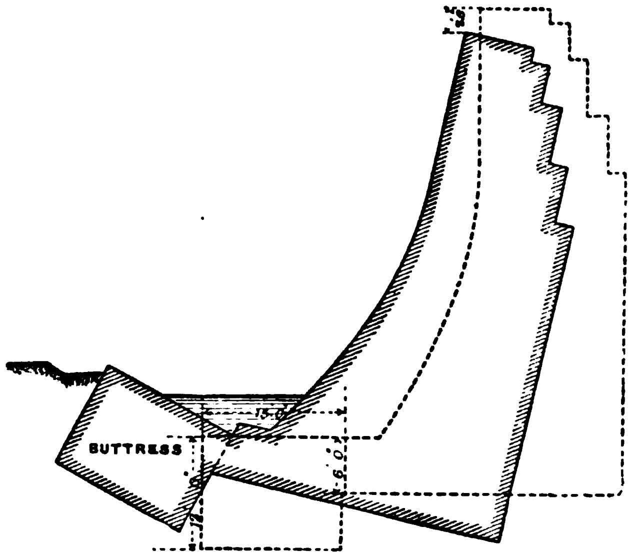
In the instance above recorded, the landslip occurred above the clay. The South-West India Dock, London, built in 1868, furnishes an example of a slip within the clay. Some portions of the dock wall were founded upon a hard bed of natural concrete, composed of gravel and shells, resting upon a layer of London clay. When the wall came to be backed up, it slid forward. In the course of excavation, for the purpose of rebuilding the wall from a deeper foundation, two disconnected surfaces of clay were found, one having slipped on the top of the other, showing that the slip had actually taken place some distance below the bottom of the wall itself.*

Another well-known instance of sliding, due to the same kind of foundation, is that of the walls of the Empress Dock, at Southampton, built in 1888. A section is given, in fig. 157, showing the position taken up by the east wall of the dock after movement. It will be noticed that the earth in front of the toe has been heaped up above its original level. The buttress shown in the figure is one of a series, each 20 feet long, 15 feet wide, and 12 feet deep, set at about 30 feet apart, with the intention of strengthening the wall after a previous experience of its weakness.†

The walls of the Kidderpur Dock, at Calcutta, have already been mentioned (p. 182, *ante*) and a section given. In one case there was a central

* *Min. Proc. Inst. C.E.*, vol. cxxi., p. 120. † *Ibid.*, p. 127.

forward projection of 7 feet 5½ inches in a length of 2,080 feet; an adjoining wall was thrust forward no less than 13 feet in a length of only 450 feet. In neither case was the deviation from the vertical of any consequence, apparently demonstrating that the slip of the backing extended to a greater depth than the foundation of the wall.* Immediately upon the occurrence of the slip, which took place during the process of backing the wall, the water was admitted to the dock, and no further movement has since been manifested. The author is personally aware of another case where the hydrostatic pressure in front of a dock wall constitutes its principal element of stability. Built, in the first place, with a view to merely temporary uses, the wall was allowed to remain in conjunction with work



Scale 20 Ft = 1 In

Fig. 157.—Dock Wall at Southampton.

of a more durable and solid character. An experimental lowering of the water in the dock, on a recent occasion, had to be abandoned owing to serious signs of failure showing themselves in the form of cracks and fissures behind the wall.

Another instance of failure, but of a different kind and somewhat puzzling as to its origin, is that exhibited in fig. 158, part of which represents the section of an old wall at the Huskisson Dock, Liverpool. Some years ago when the wall came to be examined it was found that a portion of the front masonry, at a depth of 15 feet below the surface level of the water, had by some means been displaced, had fallen out and was then lying in the dock bottom. The length affected was about 400 lineal feet, the disturbance varying from a crack to the maximum gap exhibited in the

* *Min. Proc. Inst., C.E.*, p. 104 *et seq.*

figure. The strange thing was that the wall showed no signs of collapsing altogether. The cause of the mischief is still obscure. An examination of the stability of the section by theoretical principles revealed no weakness. Apparently had there been excessive compression on the face, the upper part of the wall, deprived of its support, should have collapsed, but this is what did not happen. The wall was repaired by a refacing of concrete, 3 feet thick, and as an improvement of the dock was in contemplation at the same time, advantage was taken of the opportunity to deepen the foundations of the wall by an operation about to be described.

Fig. 158.—Dock Wall at Liverpool. Fig. 159.—Underpinning at Ardrossan.

Underpinning.—Occasionally an engineer has to face the problem, not of constructing a new wall, but of adapting an old one to conditions far other than those contemplated at the time of its construction. A common instance is that in which it is requisite to deepen an existing dock in order to accommodate vessels with greater draught. This necessitates a corresponding lowering of the quay wall and its foundations, a process called underbuilding or underpinning.

When the work can be carried out in the open—that is, with the dock run dry—it is attended by no more than the usual difficulties, though much, of course, depends on the nature of the strata to be undercut. More arduous and less secure is the operation when it has to be performed with the dock under normal conditions.

Fig. 159 is a section of a quay wall of Eglinton Dock, Ardrossan, to which the following extract refers :— *

* Robertson on "Ardrossan Harbour Extensions," *Min. Proc. Inst. C.E.*, vol. cxx.

"The portions of the north and south walls of the old tidal harbour, extending along the side of the new dock, were retained, but having been founded on clay they were underbuilt to the rock with rubble concrete, to a depth varying between 2 feet and 20 feet. The clay below the walls was excavated back 5 feet from the face of the wall, and the front of the wall was supported from the rock by raking shores. The rubble concrete underbuilding has a uniform thickness of 5 feet, where the depth is less than 10 feet, but for greater depths the underbuilding is 6 feet thick at the top, increasing downwards with the batter of the wall. The excavation was taken out in alternate lengths of about 10 feet, and the clay left between until the blocks on each side were thoroughly set; then the intermediate

Crane Road

Rock

Fig. 160.—Underpinning at Liverpool.

spaces were excavated and built up. The rubble concrete was built in layers of about 18 inches or 2 feet, until too close to the underside of the old walls for men to go in below; it was then built from the front and the concrete carefully rammed into the back. When the concrete was within 1 inch or 2 inches of the underside of the wall, an additional board, bevelled outwards, was put in the front of the frame; liquid grout of cement and sand was poured in, filling up the small space between the concrete and the wall. This proved most satisfactory, as shown by an examination of the side of each block on excavating the intermediate space. No trouble was experienced in supporting the walls, and no settlement took place during the underbuilding."

Equally satisfactory, though attended by more risk, has been the result

of underpinning operations as carried out at certain of the Liverpool docks. Owing to the exigencies of traffic the work had to be done in sections, with the dock full of water, so as to interfere as little as possible with shipping accommodation. Fig. 160 shows a section of the old wall, at the commencement of the underpinning, and fig. 161 is a section of the completed undertaking. It will be observed that the work was carried on under cover of a sheeting dam, described elsewhere (p. 105, *ante*), strutted and shored to the old wall, at a distance of about 17 feet. Below the level of the dock bottom, an inner trench was excavated between two rows of sheeting piles, one of which was situated at the extreme back of the wall and the other in front of it. Within these limits the underpinning was effected on similar lines to the underbuilding at Ardrossan. The bays were from 10 to 15 feet in length and were dealt with singly, the work being attacked at several points simultaneously. The new work consisted entirely of 6 to 1 concrete, carefully tongued into the old masonry, the surface of which was well washed and picked rough. When the concrete had been deposited to within 3 feet of the underside of the existing base, the remaining layer was put in, in three sections, advancing from the back towards the front, behind roughly constructed barriers of rubble, the concrete being carefully rammed tight and the whole grouted.

Fig. 161.—Dock Wall as Underpinned.

Miscellaneous Types of Wall.—It will be as well to conclude the chapter with some miscellaneous examples of the very varied range of types to be found among dock walls. Figs. 162 to 164 are plan and sections of the Albert Dock wall at Hull,* or, rather, the wall as originally designed and only executed for a part of its length, owing to modifications introduced as the work proceeded. This type of wall with an arched front is unusual, and it has obvious inconveniences, though as regards its structural qualities, a broad base with a minimum of masonry was held to counterbalance these drawbacks on a foundation which was incapable of sustaining much pressure. A similar type of wall, consisting of alternate piers and arches, is to be found at Bordeaux.

The sections (figs. 165 and 166) of two dock walls at Greenock are self-explanatory and do not call for any remarks, except that it may be well to add that the quarry refuse filling behind the western tidal harbour

* Hawkshaw on "The Albert Dock, Hull," *Min. Proc. Inst. C.E.*, vol. xli.

wall was washed in with Portland cement in the proportion of 1 to 15, as high as low-water level.*

ARCHED WALL ELEVATION AND SECTION.

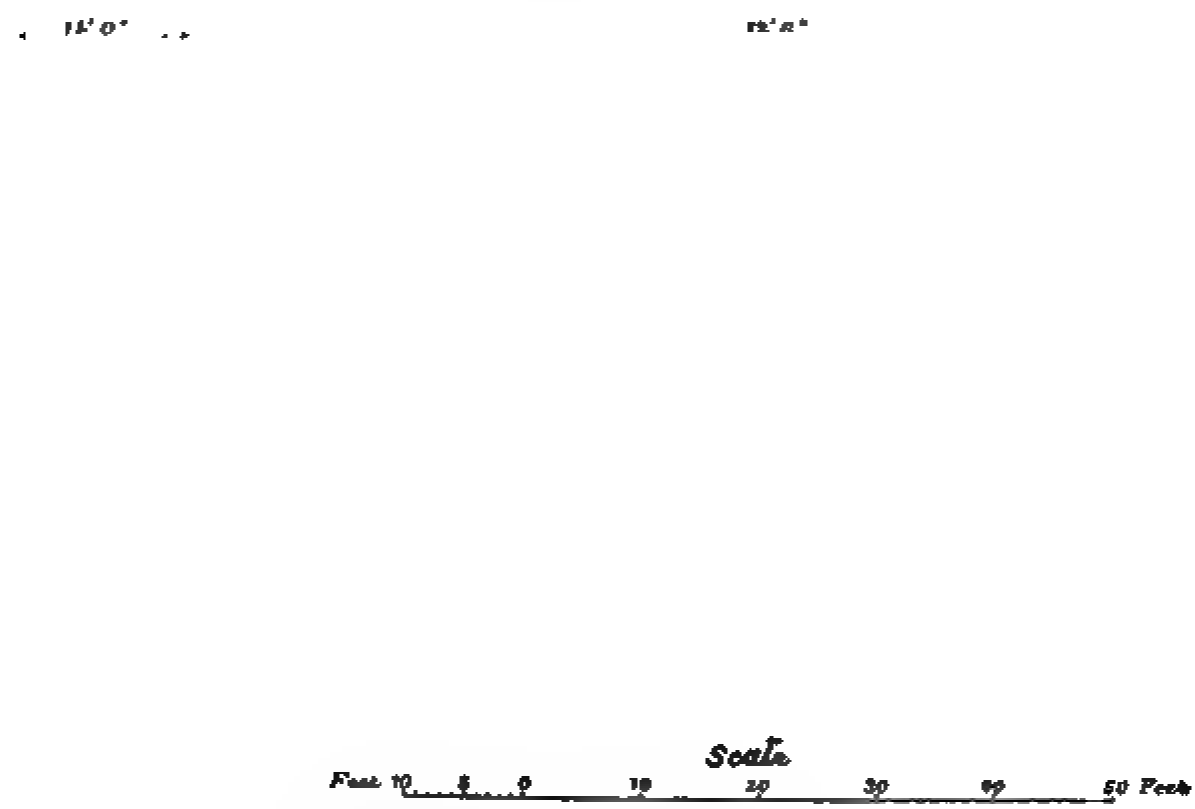
ARCHED WALL SECTION.

ARCHED WALL PLAN.

Figs. 162, 163, and 164.—Dock Wall at Hull.

Fig. 167 is a section of the Alexandra Dock wall at Hull. Originally designed to be constructed with an ashlar stone face and rubble chalk back-

* Kinipple on "Greenock Harbour," *Min. Proc. Inst. C.E.*, vol. cxxx.



Figs. 165 and 166.—Dock Walls at Greenock.

Fig. 167.—Dock Wall at Hull.

Fig. 168.—Quay Wall at Tilbury.

ing up to 14 feet below coping, a strike of masons led to the substitution of Portland cement concrete. The upper part of the wall, 14 feet in height, was built as designed with ashlar facing, projecting 6 inches to form a fender, and with granite coping. The weep-holes are at 75 feet intervals.*

The section of the tidal basin wall at Tilbury Docks, London, is given in fig. 168. The material used for the bulk of the wall was concrete, composed of 10 parts of ballast to 1 of Portland cement. The concrete work was faced above low-water mark with blue bricks, having a stock brick backing—the whole being 9 inches in thickness, with half brick piers, about 4 feet apart, dovetailing into the concrete.†

The latest type of Liverpool wall (fig. 169) is built entirely of concrete,

Fig. 169.—Dock Wall at Liverpool.

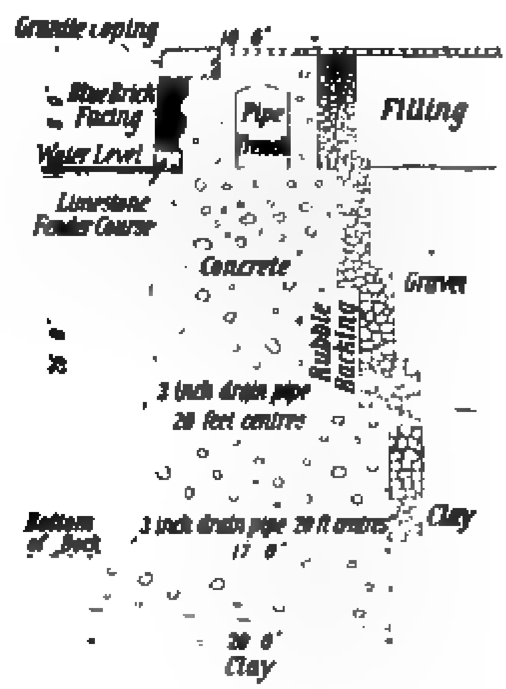


Fig. 170.—Dock Wall at Manchester.

with the exception of a granite coping. The hearting is composed of 8 parts of gravel to 1 of Portland cement, with as many burrs or plums of clean rubble and old masonry as can conveniently be bedded in. The facing, 12 inches thick, is of 6 to 1 concrete without burrs.

The new wall for the extension of the Manchester Docks is also mainly composed of concrete (fig. 170). It has a blue brick facing above water level, surmounting a limestone fender course. The coping is of granite.

* Hurtzig on "The Alexandra Dock, Hull," *Min. Proc. Inst. C.E.*, vol. xcii.
† Scott on "The Tilbury Docks, London," *Min. Proc. Inst. C.E.*, vol. cxx.

REFERENCE WORKS.

The following are a few of the sources from which the student may obtain additional information on the vexed question of earth pressure against retaining walls :—

"The Actual Lateral Pressure of Earthwork." By Sir B. Baker. *Min. Proc. Inst. C.E.*, vol. lxv., p. 140.

"The Slopes of Cuttings." By Wilfrid Airy. *Min. Proc. Inst. C.E.*, vol. lv., p. 241.

"Theory of the Stability and Pressure of Loose Earth," in *A Manual of Civil Engineering*. By Professor J. W. M. Rankine. 18th edition, p. 318.

"Essai théoretique sur l'équilibre des massifs pulvérulents, comparé à celui des massifs solides ; et sur la poussée des terres sans cohésion." By Professor J. Boussinesq. Abstract in *Min. Proc. Inst. C.E.*, vol. li., p. 277.

"Earth Pressures on Retaining Walls." By G. C. Maconchy. Article in *Engineering*, vol. lxvi., p. 256.

"Dock Walls." By J. R. Allen.

"Some Experiments on Conjugate Pressures in Fine Sand." By G. Wilson. *Min. Proc. Inst. C.E.*, vol. cxlix.

CHAPTER VI.

ENTRANCES, PASSAGES, AND LOCKS.

General Aspects of the Subject—Site—Effect of Wind, Wave, and Current—Direction—Size—Draught of Water in Approach Channel—Arrangement and Types—Simple Entrances, Locks, and Half-tide Basins—Maintenance of Fairway—Sluicing—Velocity of Efflux—Friction of Culverts—Coefficients of Discharge—Sluicing Arrangements at Liverpool, Ostend, Honfleur, Ramsgate, Dover, and Dublin—Scraping and Scuttling—Dredging—Lock Foundations—Boils and Springs—Instances at Hull and Liverpool—Suggestions for Treatment—Grouting—Stock-ramming—Sand Concrete—Lock Construction—Sills—Platforms—Recesses—Walls—Culverts—Penstocks or Cloughs—Stoney Sluices—Fan Gates—Pivotted Gates—Duration of Levelling Operations—Examples of Dock Entrances at Liverpool, Dunkirk, Buenos Ayres, Kidderpur, Eastham, Barry, Ardrossan, Hull, and Bremerhaven.

General Aspects of the Subject.—The subject of dock entrances is one demanding the most careful attention, seeing that the utility and value of an entire dock system depend to a very large extent, if not mainly, upon the safety and accessibility of its entrances.

If the docking and undocking of ships could be carried on invariably in calm weather, and with smooth water, many of the most acute difficulties of the problem would at once disappear. But ships have to be docked in foul weather as well as fair, and, apart altogether from the desirability of their obtaining shelter at the earliest possible moment from rough winds and tempestuous seas, there is the more cogent reason that the exigencies of modern commerce will not allow of a ship missing her berth in dock by one hour more than is absolutely necessary for her actual voyage; neither will they admit of her failing to leave her berth at the specified time. Every hour of extra detention in port represents to her owners a large sum in wages, maintenance and interest, unprofitably expended. Consequently, it becomes a qualification of the highest importance for a dock entrance to be available at all times and under all conditions. It must certainly be admitted that, as yet, many commercial seaports are unable to comply with this requirement, owing to obstacles arising from natural causes, such as an extreme range of tide, a shallow bar, strong currents, and sudden floods. But it is increasingly evident that the qualification will ultimately become the *sine quâ non* of a flourishing port. At the present time extensive operations are in progress at various places, notably at Liverpool, with the object of increasing the period of accessibility and eventually of transform-

ing an intermittent into a continuous service. In the dredging of bars, the lowering of dock sills and floors, two of the main obstacles to the ideal condition can be artificially overcome, and the problem then simply resolves itself into a question of fixing a judicious limit to the expenditure incurred, so as to achieve the most beneficial result commensurate with the port's resources and prospects.

Docks in tideless seas, as the Mediterranean ;* in inland situations, as at Rouen and Bremen ; and in localities where there is only a small range of tide, as is the case at Glasgow and Southampton, are endowed by Nature with signal advantages in this respect, which enable them to dispense with all the costly apparatus necessary for periodically closing their entrances, together with all the time and labour involved in the operation, while, at the same time, it confers upon them special facilities for the prompt reception and discharge of shipping. On the other hand, such docks reproduce every fluctuation of the external water level, and from the very continuity of their systems, their entrances are liable to constitute quiescent depositing areas for silt and detritus, brought in by passing currents. This last drawback, however, is one from which tidal ports themselves are not altogether exempt.

In determining the dispositions to be adopted for a dock entrance, the following points have primary importance, viz. :—(1) Site ; (2) direction ; and (3) size.

Site.—The site should obviously be the most sheltered spot available for the purpose. Exposure during docking operations to the direct influence of even a moderate gale may render a vessel temporarily unmanageable, and cause her to drift into situations dangerous alike to herself and to neighbouring craft. The writer has seen several lineal yards of granite coping at a dock entrance detruded by the stem of a vessel, under no way, but imperfectly controlled, while docking in a heavy swell. The strain upon entrance gates at such times is likewise exceedingly great, especially immediately after they have been closed. Until sufficient head is acquired on the inner side, by the fall of the tide, to keep them fairly mitred, the leaves are undergoing a series of chafings and concussions against one another and the sill, and even when actual movement in them has ceased, they are still dynamically stressed by the impact of breaking waves.

For this reason direct communication—and by this is meant communication in an uninterruptedly straight line—with the open sea is to be avoided, wherever practicable. In the case of ports on the seaboard, outer harbours or entrance channels should be provided of length, at least, sufficient to admit of a vessel losing the way which she may have gathered in making for her destination under stress of weather. The length of sheltered reach necessary for this purpose will vary with particular circumstances, but the following instances may be cited as

* The Mediterranean is not strictly tideless, but the range of tide is so small as to be negligible.

generally representative of practice in this respect:—Barry has an entrance channel, between breakwaters, 470 yards long. At Leith, a similar channel extends to 660 yards. Sunderland has an enclosed outer harbour affording a run of 900 yards. At Dover the present protected length is 750 yards, but when the new works, now in progress, are completed there will be a sheltered reach of at least 1,100 lineal yards within the breakwaters.

The objections attending a sea-exposed entrance are, of course, equally potent in the case of ports situated on broad river estuaries, flanked by low-lying country. Though the river mouth may be, to a certain extent, considered as supplying the functions of an entrance channel, yet it is often found expedient to provide a vestibule to the docks, in the form of a tidal basin, having free communication with the river. This is the plan adopted at the Liverpool and Birkenhead Docks, the Tilbury Docks at London, the Royal Dock at Grimsby, and others. The Canada Basin at Liverpool has an entrance width of 390 feet and a water area of 9½ acres. The North Basin at Birkenhead has an entrance width of 500 feet and a water area of 4½ acres, while at the Tilbury Basin the entrance is 364 feet wide and the water area 17½ acres.

In wide estuaries sheltered by ranges of hills, and narrow estuaries generally, in land-locked bays and lagunes, and on inland rivers, the foregoing precautions are rendered unnecessary, except for other and purely local reasons.

The three natural agencies influencing the eligibility, or otherwise, of a site for a dock entrance are (*a*) wind, (*b*) wave, and (*c*) current. It will be well to subject them to a brief consideration.

For the purpose of the present section we need not investigate the effect of these natural forces except in so far as they favour or interfere with the effective use of entrances, and the manipulation of vessels. Any inquiry in regard to their action upon permanent structures will be deferred until we come to the chapter dealing with the parts most affected—viz., jetties, wharfs, and piers.

Wind.—The power exerted by the wind is often sufficient to greatly impede, if not absolutely prevent, the manœuvring of vessels (more particularly those with a high freeboard), into and through a narrow, exposed waterway. The effect is greatest when the direction of the wind is broadside on, causing the vessel to fall off to the leeward. A head wind can always be counteracted by adequate tractive or propelling power; in a side wind this is of no avail, and the vessel has to be kept in her course by means of ropes. Occasionally accidents happen through the breaking of these ropes from excessive strain. Cases have occurred in which all the retaining ropes to a vessel have snapped in quick succession, leaving her entirely helpless. It is to be regretted that, at the present time, there is so little reliable evidence in regard to the actual pressure exerted on large surfaces by air in motion. Records have, indeed, been obtained showing very great

pressures, but the area affected has been comparatively trifling, and it is tolerably certain that the intensity of pressure registered by a small anemometer can, in no wise, be considered representative of surfaces of indefinite extent. Eminent authorities are inclined to take this view, and Sir John Wolfe Barry, in his Presidential Address to the Mechanical Section of the British Association meeting, in 1898, pointed out that of two wind gauges of 300 and 1·5 square feet respectively, at the Forth Bridge, under the same conditions of wind and exposure, the larger registered a pressure of 38·7 per cent. less per square foot than the smaller, while of two other gauges with more greatly contrasted areas, at the Tower Bridge, the divergency amounted to over 70 per cent. Prior to the Tay Bridge disaster, in 1879, the recognised maximum allowance for wind pressure, in Great Britain, on exposed surfaces, was 40 lbs. per square foot. Acting under the influence of public opinion, the Board of Trade, in 1880, raised the safe limit to 56 lbs., at which figure—an undoubtedly excessive one—it now stands.

The following table shows the ratio of wind pressure to velocity, as originally published by Smeaton in the *Philosophical Transactions* of 1759, and as recently modified by Mr. W. H. Dines after a long and exhaustive series of experiments.* Taking the pressure, P, in lbs. per square foot, and the velocity, V, in miles per hour, Smeaton and Dines' formulæ are—

$$P = \cdot 00492\ V^2$$

and

$$P = \cdot 003\ V^2,$$

respectively:—

TABLE XVIII.—Force of Wind in Lbs. per Square Foot.

	Velocity in Miles per Hour.											
	10	20	30	40	50	60	70	80	90	100	110	120
Smeaton, .	·5	2·0	4·4	7·9	12·3	17·7	24·1	31·5	39·8	49·2	59·3	70·8
Dines, . .	·3	1·2	2·7	4·8	7·5	10·8	14·7	19·7	24·3	30·0	36·6	43·2

The connection, however, between velocity and pressure is one which cannot be exactly determined by a simple coefficient, and all such expressions must inevitably give results more or less erroneous, except within the narrow experimental limits upon which they are founded.

To obtain immunity for an entrance from gales blowing from all points of the compass is, of course, a manifest impossibility, but something may be done towards minimising the effect of the more noxious winds. Advantage should be taken of any natural features—headlands, promontories, and the like—or even of moderately high ground in order to secure a leeward

* Vide *Engineer*, Nov., 1897.

position. Where this is not available, artificial shelter may be created in the form of parapet walls, wind screens, and buildings generally. Offices, huts, and sheds, for the use and shelter of the dock master and his staff, may be grouped at the more exposed points so as to break the force of the wind.

The wind which blows with greatest frequency at any place is usually termed the *prevailing wind*, but it does not by any means follow that it is the wind attended by the most disastrous results. More harm may be done by a single gale from an unusual quarter than by a whole twelvemonth of the prevailing wind. In this, as in all other matters, it is necessary to acquire locally the fullest information possible.

Waves.—The action of waves, apart from tidal waves, depends, primarily, of course, upon the wind, but, once agitated, the sea maintains a momentum which may, and usually does, outlast the duration of the wind itself so as to constitute an entirely distinct source of activity.

The inception of waves being due to the wind, their development will largely depend upon the amount of surface acted upon. The greater the length of open sea, *ceteribus paribus*, the higher the wave which breaks upon the shore, provided always there is sufficient depth of water to admit of its formation. Intervening shoals will break up a wave, so that the effective length of sea may be much less than the apparent length.

This length, or distance within which the wave attains its development, is termed the *fetch*, and Stevenson has devised an empirical formula from which the probable height of waves may be estimated. Taking H as the height of the wave in feet, and L as the length of fetch in miles, it has been found that approximately—

$$H = 1{\cdot}5\sqrt{F}; \qquad . \quad . \quad . \quad . \quad . \quad . \quad (\alpha)$$

or, for short fetches, less than 30 miles,

$$H = 1{\cdot}5\sqrt{F} + (2{\cdot}5 - \sqrt[4]{F}). \qquad . \quad . \quad . \quad (\beta)$$

In Table xix. are one or two examples of the height of waves as deduced by the formulæ and as determined by actual observation.

The maximum fetch alone cannot, however, be considered as a criterion of the exposure of an entrance. The severest gales may not blow from that particular quarter of the compass, and, on the other hand, heavy rolling seas may be deflected so as to bring their influence to bear upon an apparently sheltered area.

In exposed situations it is possible, by artificial means, to cause a wave to spend its force before reaching the spot where its unchecked onset would be dangerous. Breakwaters, either in the form of parapet walls or as submerged mounds, may accordingly be employed to reduce the amount of fetch and to provide areas of comparative quiescence. These works, however, form a distinct branch of harbour engineering which cannot be entered upon here.

TABLE XIX.

Place of Observation.	Length of Fetch. Nautical Miles.	Observed Height of Waves, in Feet.	Calculated Height. Formula. (α)	Calculated Height. Formula. (β)
			Feet.	Feet.
Scalpa Flow,	1·0	4·0	1·5	3·0
Firth of Forth,	1·3	1·8	1·8	3·2
Lough Foyle,	7·5	4·0	2·5	3·75
Clyde,	9·0	4·0	4·5	5·25
Colonsay,	9·0	5·0	4·5	5·25
Lough Foyle,	11·0	5·0	5·0	5·7
Anstruther,	24·0	6·5	7·5	7·7
Lake of Geneva,	30·0	8·2	8·2	8·37
Buckie,	40·0	8·0	9·55	...
Douglas, I.O.M.,	65·0	10·12	12·0	...
Kingstown,	114·0	15·0	16·0	...
Sunderland,	165·0	15·0	19·3	...
Peterhead,	400·0	22·6	30·0	...

TABLE XX.

Date.	Tide	Direction of Wind.	Velocity of Wind in Miles per Hour.	Observed Height of Waves from Trough to Crest, in Feet.	Largest Vessel Docked or Undocked.	Length, in Feet.	Draught, in Feet.	No. of Tugs.	Remarks.
1902.									
Aug. 20	P.M.	N.W.	18	3½	"Majestic."	565	26½	4	Undocked without trouble.
Sept. 2	A.M.	S.W.	23	4½	"Teutonic."	565	27½	5	Do.
,, 16	A.M.	S.W.	24	5	"Workman."	450	18	2	Attempted to undock, but failed.
,, 17	A.M.	W.N.W.	20	3½	,,	450	18	2	Locked out 3 hours before H.W.
,, 17	A.M.	W.N.W.	24	4½	"Majestic."	565	25¼	...	Left at H.W. 20 minutes in basin.
Oct. 15	A.M.	W.S.W.	15	3	...	...	...	...	...
,, 15	P.M.	S.W.	24	5	...	...	...	...	...
,, 16	A.M.	N.W.	21	4¾	"Turcoman."	450	23½	2	...
,, 16	P.M.	N.W.	25	5½	None of importance.	...	...	...	Three small steamers.
,, 17	A.M.	N.W.	20	4½	"Saxonia."	600	24	4	Docked at H.W.
Dec. 18	P.M.	W.N.W.	32	5½	None of importance.	...	...	...	Two small steamers.
,, 19	A.M.	W.N.W.	25	5	"Bavarian."	500	26	4	Undocked at H.W.
,, 25	P.M.	W.S.W.	20	4½	...	...	...	...	...
,, 29	P.M.	W.	35	5½	...	...	...	...	...
,, 30	A.M.	N.W.	20	4	...	...	...	...	...

It has been stated that 2 feet is the greatest height of waves consistent with the safe working of dock gates.* The writer's experience convinces him that this estimate is too low, for he is acquainted with instances in which the gates of exposed entrances have been worked without difficulty in waves of at least twice that height. Furthermore, vessels have safely weathered the pierhead of an entrance lock with a rise and fall, due to surging, of 7 or 8 feet in the level of their decks. It may be said that while no definite limit can be fixed as the point at which the working of an entrance becomes dangerous, the practicability, or otherwise, of docking operations will largely depend on the tug and capstan power available, on the strength of the ropes and hawsers employed, and, above all, on the skill and capability of those who superintend and carry out the necessary manœuvres.

For the record (Table xx.) of noteworthy conditions during a period of four months, at the Canada Basin entrances, Liverpool, the writer is indebted to the Dockmaster, Captain Parkes.

Current.—In contradistinction to the intermittent character of the previous agencies, the third is continuous and cumulative in action. To the influence of the littoral current is due the maintenance or closing of the fairway of an entrance. Currents arise from several causes and their workings are often complex and conflicting. At one period of the day the tidal current will predominate in a river and cause an inward flow, at another it will reverse its direction, augmented by the fluvial current. At different stages of the tide there will be zones of slack water, counter-currents, and eddies. It is no uncommon feature for the tide to be flowing into the mouth of an estuary at one side while it is ebbing on the opposite shore. The course of a river is never straight, and the current is greater at the concave side of each bend than at the convex side. Hence it is that currents are perhaps the most erratic and least understood of all aqueous agencies.†

In tidal estuaries, just about the time at which the tide reaches its highest and lowest levels, there are periods of slack or still water, in which matter, hitherto kept in suspension by the movement of the current, is deposited. If allowed to accumulate in the vicinity of an entrance, the silt thus formed becomes a danger to navigation. It may, possibly, be removed by a succeeding current; if not, it will be necessary to remove it either by dredging, scouring, or sluicing.

* *Encyclopædia Britannica*, 9th ed., Art. "Harbours." It is not quite clear whether the measurement is from trough to crest or merely above mean water level. The author assumes the former.

† Lord Kelvin is reported to have said to a Parliamentary Committee, in reply to an enquiry respecting his investigation into the probable effect of certain works upon tidal currents, that he had considered the question seriously, had made many calculations, and was quite unable to arrive at any satisfactory result. *Vide* Farren on the "Silting of Small Harbours," *Min. Proc. Liverpool Engineering Society*, vol. xviii., p. 226.

As illustrative of the variation in the amount of material carried in suspension by tidal rivers the following table is inserted :— *

TABLE XXI.—SHOWING AMOUNT OF MATERIAL IN SUSPENSION IN 1 GALLON OF MERSEY WATER AT VARIOUS TIMES OF THE TIDE.

Flood Tide.		Flood Tide.		Ebb Tide.		Ebb Tide.		Ebb Tide.	
A.M.	Grains.	A.M.	Grains.	P.M.	Grains.	P.M.	Grains.	P.M.	Grains.
8.45	7·0	11.15	12·95	1.15	5·25	3.45	19·95	6.15	30·8
9.0	8·4	11.30	12·95	1.30	5·25	4.0	13·65	6.30	30·45
9.15	20·3	11.45	15·05	1.45	6·3	4.15	1·05	6.45	54·25
9.30	22·92	12.0	15·75	2.0	3·5	4.30	5·6	7.0	43·05
9.45	20·12			2.15	1·75	4.45	14·0	7.15	38·85
10.0	24·85	P.M.		2.30	2·8	5.0	34·65	7.30	46·55
10.15	24·15	12.15	12·25	2.45	2·8	5·15	18·9	7.45	52·5
10.30	23·62	12.30	10·85	3.0	10·85	5·30	25·2	8.0	32·2
10.45	17·5	12.45	9·8	3.15	5·95	5.45	30·8	8.15	46·55
11.0	21·7	1.0	9·8	3.30	8·4	6.0	22·05	8.30	38·5

The sediment in the River Hooghly, at time of flood, amounts to 3 inches per cubic foot. In the River Plate it is 1 in 10,000 by weight.

Great variation is to be found in the rate at which silting takes place. The quantity which collects in the Tilbury tidal basin is stated to be $1\frac{1}{2}$ to 2 inches daily. At Avonmouth Dock entrance, the accumulation amounts to only 15 inches per month.

Purely maintenance dredging at some ports reaches very high figures. At Kidderpur, it is 37,000 cubic yards per annum; at Bordeaux, 380,000 cubic yards; at Ostend, 500,000 cubic yards; at Hull, 830,000 cubic yards; and at Glasgow, 870,000 cubic yards per annum.

The power of currents to disturb deposited material may be gauged from the following table which indicates the critical velocity, or the velocity at which moving water just begins to exert its erosive power. To

TABLE XXII.

Material.	Critical Velocity.
Silt, mud, very soft clay, ...	3 inches per second.
Fine sand, loam, ...	5 ,, ,,
Ordinary clay, ...	6 ,, ,,
Coarse sand, fine gravel, ...	7 ,, ,,
Fairly coarse gravel, ...	12 ,, ,,
Coarse ballast (1-inch pebbles), ...	2 feet ,,
Large shingle ($1\frac{1}{2}$-inch pebbles), ...	3 ,, ,,
Heavy shingle, broken stone, ...	4 ,, ,,
Soft rock, ...	5 ,, ,,

* A. G. Lyster on "Manchester Ship Canal," *Min. Proc. Liverpool Engineering Society*, vol. vii.

retain in suspension and transport material, the current will have to exceed this limit, and, in some cases, to be very much greater.

The figures in Table xxii. relate to the bottom or bed velocity, which, according to Professor Rankine, varies between $\frac{3}{4}$ and $\frac{1}{2}$ of the surface velocity.

A moderate current in the fairway of an entrance is a desideratum from more points of view than one. It prevents silting and it assists in the manœuvring of vessels. For this reason it will be advisable to locate an entrance in the vicinity of a concavity in a river's bank rather than at a convexity. But the question is somewhat too complicated for generalities, and the engineer will have to rely largely upon his own judgment, aided by such local information as he is able to procure.

Direction.—Having determined the site, the next point to be settled is the direction of the entrance. There are three main directions in which an entrance may point—viz. (*a*) down-stream, (*b*) up-stream, and (*c*) amidstream, or at right angles to the direction of flow.

(*a*) *A down-stream entrance* is not convenient for vessels entering on a flood tide. The way on a ship is maintained or increased by the tidal flow, and effective control is more difficult. It is better, for purposes of navigation, to dock or undock a ship against the tide or current. Hence such an entrance would only be suitable for vessels docking after high water or undocking before high water. In non-tidal rivers, or those portions unaffected by the tide, the circumstances are in favour of a down-stream entrance, especially if the current is at all strong.

(*b*) The advantages and disadvantages of *an up-stream entrance* are the converse of those appertaining to a down-stream entrance. There is the additional consideration that an up-stream entrance is more likely to be silted up by detritus brought down by the river and deposited in the mouth of the entrance.

(*c*) An entrance pointing amidstream is at once the least convenient and the most convenient form for general purposes. In itself it offers grave drawbacks to navigation, for the moment a vessel's bow comes within its shelter, the unprotected stern will be swung round by the force of the current, unless it exceptionally happens to be dead high water at the moment; but if it be provided with a bell-mouth, or with trumpet-shaped wing walls, this drawback is overcome and the entrance becomes available for both ebb and flood tides, since a vessel may thus gain the leeward of either of the wing walls for her entire length before engaging in the entrance proper.

Dimensions.—The dimensions to be assigned to an entrance will obviously be regulated by the size of the largest ship frequenting the port, with a due allowance for future increment.

Half a century ago, under the régime of paddle steamers, entrances and locks had to be constructed of very considerable *widths*. When, in process of time, screws and propellers displaced paddles, the necessity for a great width of waterway temporarily disappeared, but with the growth and

development of ocean leviathans in recent years, the need of wide entrances is returning. In 1857 the Canada Lock was constructed at Liverpool, 100 feet wide. It was not until the year 1902 that another entrance of the same width was opened for traffic. During the interval the width considered requisite had fallen to 65 feet, from which it has gradually risen to its former dimension. No doubt a width of 100 feet is in excess of present-day requirements, the maximum breadth of a ship being as yet 70 feet, but another decade will probably see a large increase, so that the margin provided is no more than prudent foresight would warrant.

Another factor involved in the determination of width is the ratio between the sectional area of the entrance and the cubic capacity of the dock, or, what is the same thing, between the width of the entrance and the area of the dock. If a dock entrance remains open for any length of time after high water, a gradually increasing current is generated owing to the fall of the tidal level outside, and the consequent discharge of the water from within the dock through a narrow passage. If allowed to continue too long the current may become so rapid as to render the closing of the gates a hazardous proceeding. The limit of safety may be considered reached when the velocity is 3 feet per second. When the dock is of considerable area it may be necessary to provide two or more entrances, as much for facilities of traffic as for the reason given above.

As regards *depth*, the sill of the old Canada Lock was such as to afford a depth of 26 feet 6 inches of water at high water of ordinary spring tides at Liverpool, and 19 feet 4 inches at high water of ordinary neaps. The latest entrances constructed at that port provide for 39 feet 2 inches and 32 feet respectively. The loaded draught of modern vessels, it is true, does not exceed about 32 feet as yet, but the greatest length consistent with that draught has now been reached, and an increment in length will necessitate a corresponding increase in depth. The obstacle to this development in depth has been the limited draught of water obtainable at the ports which the vessels frequent, and there can be no doubt that with increased depth of water there will come increased depth of ships. The following abridged remarks of Dr. Francis Elgar,* made in 1893, are equally applicable at the present date :—

"The deep draught of water is a most important element of speed at sea, and it is now strictly limited by the depth of water in the ports and docks used by the fast passenger steamships on both sides of the Atlantic. The result is that it is only a question of time, and not of a very long time with our present materials of construction and type of propulsive machinery, to find an absolute limit of speed imposed by the restriction of draught of water. The Atlantic trade is increasing at such a rapid rate that larger and swifter ships are certain to be soon called for; but much deeper harbours and docks will be required if further great increases of speed at sea are to be obtained without excessive difficulty and cost."

* Elgar on "Fast Ocean Steamships," *Min. Proc. Inst. N.A.*, 1893.

Commenting on and emphasising this statement in 1898, Dr. Elmer Corthell* added the following rider:—

"It may be stated as a fact, palpable and undoubted, that no port of the world will, in the near future, be classed or used as a first-class port which will not readily admit steamers drawing at least 30 feet of water. This means 35 feet in the entrance channels through sea-bars, 32 feet in river channels and other entrance approaches, and 31 feet in harbours, basins, and along the quays and wharves."

The *length* to be given to an entrance will depend upon its arrangement, either as a lock with two pairs of gates, or as a simple entrance with one pair. In the latter case, apart from the wing walls adopted for entrances pointing amid stream, the length need not be more than will accommodate the gates and their side recesses. The length to be given to a lock entrance will, of course, be governed by the length of vessel which the lock is intended to receive. The largest lock on the Thames is the Tilbury entrance lock, 700 feet long, followed by the northern entrance lock of the Albert Dock, 550 feet long. The largest lock at Liverpool is 602 feet long; at Barry there is a lock 647 feet in length; at Barrow, 700 feet; and at Cardiff, 800 feet. This last represents the maximum length yet obtained. The new lock at Bremerhaven is 705 feet long. Swansea has an 800-feet lock in hand.

Arrangement and Types of Entrances.—Following local dispositions and requirements, there are three varieties of dock entrance, which are used either singly or in combination, viz:—

(1) A simple entrance, provided with one pair of ebb-gates.

(2) A lock, with at least two pairs of ebb-gates.

(3) A half-tide basin, intervening between the river and the dock and separated from each by a pair of gates.

Referring to these *seriatim*, it may be remarked that (1) a simple entrance is only available for navigation at or about the time of high water. Where the rise and fall in the tide is sufficient to necessitate the use of gates, the working period will generally be confined to a period of three hours, or less, in each tide. Furthermore, a single pair of gates is but inadequate provision against contingencies. Should an accident by any means happen to the gates so that they could not be closed, a very grave risk would be incurred. The unexpected running dry of a dock would probably cause irreparable damage to the shipping berthed within it.

(2) A lock offers additional facilities for the docking of vessels, since it can discharge its functions for some time after the water within the dock has been impounded; to be precise, as long as there is sufficient depth of water on the outer sill to admit of boats entering the lock. It is a particularly useful arrangement when the dock is frequented by barges, lighters, and other small craft; and its value is enhanced by dividing the lock, by

* Corthell on "Maritime Commerce," *Min. Proc. American Association for the Advancement of Science*, vol. xlvii.

means of a pair of intermediate gates, into two sections or lengths, so that it can be accommodated to the reception of large or small vessels, as the case may be, with the minimum expenditure of water during the process. The quantity of water withdrawn from the dock will be a matter for consideration if the operation of locking be very prolonged. The following table, modified from one in Rankine's work on *Civil Engineering*, shows the results of lockage under various conditions.

Let L denote a lockful of water—that is, the volume contained in the lock chamber, between the upper and lower water levels; let B denote the volume displaced by a boat.

TABLE XXIII.—LOCKAGE.

	Lock Found	Water Discharged.	Lock Left
One boat undocking,	Empty.	L − B.	Empty.
,, ,,	Full.	...	Empty.
,, docking,	Empty or full.	L + B.	Full.
Two *n* boats docking and undocking alternately,	Undocking, full. / Docking, empty.	*n* L.	Undocking, empty. / Docking, full.
Series of *n* boats undocking,	Empty.	*n* L − *n* B.	Empty.
,, ,, ,,	Full.	(*n* − 1) L − *n* B.	Empty.
,, ,, docking,	Empty or full.	*n* L + *n* B.	Full.
Two series, each of *n* boats, the first undocking, the second docking,	Full.	(2 *n* − 1) L.	Full.

Against the advantages afforded by the use of a lock have to be set the greatly increased cost of construction over that of a simple entrance and the additional space required. The projection of the inner end of a lock into the dock itself is a plan which, though often adopted, is attended by a decrease in the utilisable length of quay and in the convenience of berthing.

In large ports, the combination of a simple entrance with one or more locks is no uncommon feature. The former is used for docking large ships during the period in which there is free communication between the dock and the river; the latter, which are often in two widths, are brought into active service when the entrance is closed, or they may be utilised contemporaneously as subsidiary entrances. At Barry there is a single entrance, 80 feet in width, and a lock adjoining, 647 feet by 60 feet. The recently constructed entrances, at the north end of the Liverpool dock system, comprise an entrance,* 100 feet wide, an 80-foot lock, 130 feet long, and a 40-foot lock, 165 feet long. These are all parallel in direction, pointing up-stream, but at Kidderpur docks, advantage has been taken of a bend in the waterway to arrange a lock, 400 feet by 60 feet, in an up-stream

* This entrance is, strictly speaking, a lock, being provided with two pairs of ebb-gates; but it is rarely, if ever, used as such, the chamber being only 130 feet long, and the provision of two pairs of gates is really a safeguard against the contingencies previously referred to.

direction, and an 80-foot entrance pointing down-stream. Ships docked before high water, anchor above the upper entrance, and, when the gates can be opened, are breasted in alongside the jetty-head. The lower entrance is intended for the use of vessels which cannot arrive before high water; it is also required during freshets in the rainy season, when the current in the river is always down-stream.

In connection with parallel entrances it has been noted, in the Mersey, that during the time in which they are open, a circulating current has been set up, the water entering through one passage and making its exit by the other, and this quite regardless of any change in the tide.

At the entrance to the Manchester Ship Canal there are three parallel locks—30 feet by 150 feet, 50 feet by 350 feet, and 80 feet by 600 feet respectively.

(3) Half-tide basins, which are practically locks on a very large scale, are said to be due to the initiative of the late Mr. Jesse Hartley. They differ only from locks in regard to their irregular shape and great size. The gates of the dock proper are closed at, or soon after, high water, whereas the gates of the half-tide basin are kept open, as the name implies, for several hours afterwards, so that belated vessels can enter as long as there is sufficient depth of water over the outer sill which, of course, is necessarily lower than that of the inner dock. Vessels may remain in the half-tide dock until the ensuing flood tide and discharge part of their cargo there, or, if it be desirable to establish immediate communication with the inner dock, this can be done by pumping water into the half-tide dock from some external supply, usually the river itself. To equalise the level by running down the water in the inner dock would generally prove to be too wasteful of water, unless the latter were relatively much larger than the half-tide basin. This last condition may, of course, be fulfilled by grouping several inner docks together. The Sandon half-tide dock at Liverpool has an area of 14 acres, and is in direct communication with the Sandon Dock (10 acres), the Huskisson Dock and branches (36 acres), the Wellington Dock (8 acres), and the Bramley Moore Dock (10 acres)—64 acres in all.* The North Dock (13 acres) at Swansea is approached by two half-tide basins, one at each end, with areas of 2½ and 1½ acres, respectively. At Sunderland there is a half-tide basin of 2½ acres, acting as a vestibule to the Hudson Docks, of over 40 acres in extent.

Maintenance of Fairway.—The absolute necessity for a sufficient and continuous depth of water in the channel leading to a dock entrance is self-evident. The tendency, which the channel has, to become silted up, must be checked by some corrective agency, either natural or artificial. The natural means would be the utilisation of some beneficial current. Where this is impracticable, recourse must be had to sluicing, scouring, scraping, or dredging.

Sluicing.—This method consists in forming an aqueduct or culvert in the side walls of an entrance, communicating with the dock at its inner end,

* And indirectly with others, the total impounded area being over 100 acres.

and branching into a series of outlets, discharging as low as possible, at convenient intervals along the channel frontage. During the lowest period of ebb-tide, water from the dock is allowed to run off through these culverts and the velocity, which it possesses in consequence of the head of water within the dock, enables it to stir up and remove the mud in front of the outlets. The quantity of water run off is controlled by a penstock, or paddle, near the entrance of the culvert, and, in addition to this, other pe stocks are often provided, one at each outlet in order to regulate the number of exits, for it may often be desirable to concentrate the whole discharge at a few points in order to obtain the maximum effect. Where this system is adopted, it is very essential to provide a masonry or concrete apron in front of the wall, otherwise there will be a decided risk of the wall becoming undermined. For the same reason the discharge should be perfectly horizontal, as any downward inclination causes the water to act the part of an excavator. The ground in such cases is ploughed up, and the excavated material is deposited a short distance away as soon as the current slackens, in such a manner as to form a ridge, which, being out of range of the sluice, is very dangerous, and can only be removed by dredging.

This tendency to excavate below the toe of a wall is one of the drawbacks of a mural sluice; another is that its effective action is restricted to a very small area immediately in front of the opening, so that it lowers the sides of the channel at the expense of the middle of the bed. A third objection lies in the fact that the formation of numerous outlets at the base of the wall weakens the wall at the locus of greatest intensity of pressure. A fourth objection is the very serious loss of head due to friction and bends, whereby the force of the discharge is materially diminished.

Accordingly, it is not surprising that the alternative method of sluicing through apertures in the dock gates has been adopted in many cases. There is an absence of skin friction, there are no bends, and the only loss of head is that due to discharge through a thin orifice, which is much less than the loss due to friction in a long conduit. Furthermore, by this means a large body of water is discharged along the axis of the channel, the bed of which is thus kept clear without endangering the stability of the wing walls. On the other hand, the provision of sluice valves and gear adds considerably to the weight of the gates and entails greater strength in their structure.

Velocity of Efflux from Sluices.—The velocity of efflux, from which the scouring effect of a sluicing current can be gauged, is calculated from formulæ based upon the following principles :—

The theoretical velocity of a liquid issuing from an outlet under a given head or charge, considered without reference to friction, is the same as that acquired by a solid particle in falling freely from a height equal to the head—*i.e.*,

$$h = \frac{v^2}{2g},$$

or,

$$v = \sqrt{2gh}. \quad . \quad . \quad . \quad . \quad (37)$$

In the case of a liquid whose motion is impeded by friction, the rate of flow is naturally less. The amount of reduction may be expressed by a fractional coefficient, attached to the preceding equation, denoting the proportion of head expended in overcoming the frictional resistance. Thus, the total head may be considered as divided into two portions, only one of which is available for producing velocity—

$$h = (1 + F)\frac{v^2}{2g},$$

whence

$$v = \sqrt{\frac{2gh}{1 + F}}. \quad . \quad . \quad . \quad . \quad (38)$$

The laws of fluid friction, which it will be useful to state at this point, differ materially from those relating to the surface-contact of solid bodies. They are as follows :—

1. The friction is independent of the head, or pressure.
2. It varies directly as the area of the surface exposed to action.
3. It varies directly (or very nearly so) as the square of the velocity. This, however, is only literally true so long as the rate of flow is sufficient to prevent the adherence of water to the surface in question.

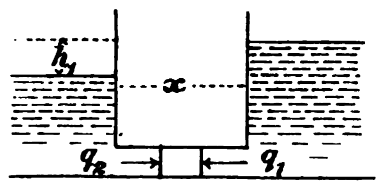


Fig. 171.

Now, let us consider the case of a horizontal culvert of length, x (fig. 171), and sectional area, a, in which water is running full. Agreeably to the foregoing laws, we may express the amount of surface friction as

$$S = f \cdot p \cdot x \cdot v^2,$$

where f is a coefficient to be determined later, and p is the perimeter of fluid section.

Now, assume the surface friction to be just counteracted by the difference of pressure upon the two faces of the length, x. That is—

$$(q_1 - q_2)\,a = f \cdot p \cdot x \cdot v^2.$$

But this resultant pressure, $(q_1 - q_2)\,a$, is due to a difference in head on each side of the culvert. Hence, we may substitute for it the expression for the pressure of the differential head—viz., $w\,h_1\,a$, in which w is the weight of a cubic foot of water. At the same time, let $R = \frac{a}{p}$, and the equation becomes

$$h_1 = f \cdot \frac{x}{R} \cdot \frac{v^2}{w}.$$

Now, the term $\frac{v^2}{w}$ (w = 64 lbs. for salt water) deviates by so little from $\frac{v^2}{2g}$ that we can replace it by the latter, without sensible error. Whence,

$$h_1 = f \cdot \frac{x}{R} \cdot \frac{v^2}{2g},$$

or,

$$v = \sqrt{\frac{R}{fx} 2g h_1}. \quad . \quad . \quad . \quad . \quad (39)$$

This value for h_1 determines the amount of head absorbed in overcoming friction. Its ratio to that given above (37) for simple discharge is expressed by the coefficient: $F = f\frac{x}{R}$. The factor, f, varies with the nature of the surface of the conduit, and it is also found to depend, to a certain extent, on the relative diameter of the conduit and the rate of flow, being greater in small pipes than in large culverts, and at low velocities than at high speeds. Its value is found, however, to lie between ·005 and ·01, and ·0075 may be taken as a serviceable mean for general use under normal conditions.

The symbol, R, standing for the area of fluid section divided by the perimeter, is referred to as the hydraulic mean radius, or the hydraulic mean depth. For circular and square culverts running full, and for circular culverts running half full, it is obviously equal to one-fourth of the diameter.

There are other sources of friction than that investigated above, and these cannot be overlooked in estimating the efficiency of the current issuing from a sluicing culvert:—

I. There is the friction due to the form of inlet at the reservoir. If an orifice in a thin plate, it has been found by experiment that

$$F_2 = \cdot 055.$$

If the inlet has a square-edged entrance,

$$F_2 = \cdot 505.$$

II. There is the friction at sudden enlargements or contractions of culvert area. Let the ratio in which the effective area is suddenly enlarged or contracted be designated r. Then, for abrupt enlargements,

$$F_3 = (r - 1)^2,$$

and for abrupt contractions the same formula may be used, although the actual ratio of contraction is somewhat uncertain, being greater than the apparent ratio. The loss of head is due to the enlargement succeeding contraction.

III. For bends in circular culverts,

$$F_4 = \frac{\theta}{\pi}\left\{0{\cdot}131 + 1{\cdot}847\left(\frac{d}{2r}\right)^{\frac{7}{2}}\right\},$$

and in rectangular culverts,

$$F_4 = \frac{\theta}{\pi}\left\{0{\cdot}124 + 3{\cdot}104\left(\frac{d}{2\,r}\right)^{\frac{7}{2}}\right\},$$

are formulæ enunciated by Weisbach, r being the radius of curvature of the centre line, and θ the angle through which the culvert is bent For very sharp turns, or knees,

$$F_4 = 0{\cdot}946 \sin^2\frac{\theta}{2} + 2{\cdot}05 \sin^4\frac{\theta}{2}.$$

The head necessary to overcome all these varied sources of friction must be deducted from the total head, and the residue will then represent the head available for producing velocity of exit, in accordance with the formula

$$v^2 = \sqrt{2\,g}\,\mathrm{H}.$$

The theoretical quantity of water discharged is

$$Q = A\,v,$$

where A is the area of opening, but in practice it is further necessary to take into account a modification due to the contraction of the free effluent leaving the culvert, by which the effective area of the current is less than the total area in a certain ratio, dependent on the shape of the outlet. This is brought about by the convergence of the particles into a *vena contracta*, or contracted vein.

Calling the pipe or culvert area unity, the following are coefficients (c) of actual discharge in the formula $Q = c\,A\,v$.

For wide openings, whose bottom is on a level with that of the reservoir; for culverts with walls in a line with the orifice, ...	·96.
For narrow openings, whose bottom is on a level with that of the reservoir, ...	·86.
For sluices, without culverts or side walls, ...	·61.

In the foregoing investigation we have only credited the fluid current with the energy due to motion and to head or pressure, this being the case when the culvert is truly horizontal. When, however, there is a fall or inclination in the culvert the water possesses another source of energy, viz., energy of position, and this leads us to undertake an investigation into the principles which govern the flow of water in inclined pipes and culverts.

Reverting to the laws of fluid friction stated on p. 239, and remembering that when motion has become uniform, the acceleration and retardation of a current neutralise each other, we can form the following equation connecting the two. The acceleration is that due to the action of gravity on a body falling down an inclined plane of height, h, and length, l. Accordingly,

$$g\,\frac{h}{l} = pv^2 \times \text{constant};$$

or, substituting S for $\frac{h}{l}$, the sine of slope, and introducing $\frac{1}{R} = \frac{p}{a}$ instead of p, so as to express the equation in terms of the hydraulic mean radius, we have

$$gS = \frac{V^2}{R} \times \text{constant},$$

which reduces to the form

$$V = C\sqrt{RS}, \quad . \quad . \quad . \quad . \quad . \quad . \quad (40)$$

and this constitutes the basis of a very large number of expressions for the velocity, the values for C ranging from 70 to 100, according to the personal observation of different experimentalists.

Kutter's value for C, though complex, is recognised as the most generally reliable, and it is here given

$$C = \frac{41{\cdot}6 + \frac{1{\cdot}811}{a} + \frac{{\cdot}00282}{S}}{1 + \left(41{\cdot}6 + \frac{{\cdot}00282}{S}\right)\frac{a}{\sqrt{R}}} \quad . \quad . \quad . \quad . \quad (41)$$

in which a has the following numerical equivalents :—

·009 for well-planed timber channel.
·010 „ cement plaster channel.
·011 „ cement and sand plaster channel.
·012 „ common boards, unplaned.
·013 „ ashlar and neatly-jointed brickwork.
·017 „ rubble masonry.
·025 „ earth surface.
·03 „ detritus and uneven ground.

Strictly speaking, the amount of head introduced into the foregoing equation should be the total head reduced by that portion required to overcome the friction of entrance into the culvert, but when this latter is very small in comparison with the former, as it is in long conduits with moderate heads, the total head may be used without sensible error.

For the sake of example let us take the case of a horizontal culvert, 6 feet high by 4 feet wide, and find the amount of head required to produce an exit velocity of 4 feet per second. Assume a length of 100 feet, a square-edged entrance, and one bend of 60° in direction, with a radius of 5 feet.

Then, by the preceding formulæ,

$$F_1 = f\frac{x}{R} = \frac{{\cdot}0075 \times 100}{1{\cdot}2} \quad . \quad . \quad . \quad . \quad = {\cdot}625$$

$$F_2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad = {\cdot}505$$

$$F_3 = \frac{\theta}{\pi}\left\{0{\cdot}124 + 3{\cdot}104\left(\frac{d}{2r}\right)^{\frac{7}{2}}\right\}$$

$$= \frac{1}{3}\left\{0{\cdot}124 + 3{\cdot}104\left(\frac{2}{10}\right)^{\frac{7}{2}}\right\} \quad . \quad . \quad = {\cdot}045$$

$$F = F_1 + F_2 + F_3 = 1{\cdot}175$$

$$H = (1 + F)\frac{v^2}{2g}$$

$$= 2{\cdot}175 \times \tfrac{16}{64} = {\cdot}544 \text{ foot, or } 6\tfrac{1}{2} \text{ inches.}$$

The head required to produce the same velocity through a simple sluice opening, as in a gate, will be as follows:—

$$F_1 = {\cdot}055.$$

$H = (1 + F_1)\dfrac{v^2}{2g} = 1{\cdot}055 \times \tfrac{16}{64} = {\cdot}264$ foot, or a little over 3 inches—about one-half of the head required in the former case.

It may be interesting to compare the foregoing problems with a kindred one calculated by Kutter's formula. Suppose the culvert, as above, to have an inclination equal to that afforded by the head—viz., $6\frac{1}{2}$ inches—or, to simplify calculation, say 7 inches in 100 feet.

$$v = \frac{41{\cdot}6 + \dfrac{1{\cdot}811}{{\cdot}01} + \dfrac{{\cdot}00282 \times 100 \times 12}{7}}{1 + \left(41{\cdot}6 + \dfrac{{\cdot}00282 \times 100 \times 12}{7}\right)\dfrac{{\cdot}01}{\sqrt{1{\cdot}2}}} \sqrt{\frac{{\cdot}12 \times 7}{100 \times 12}}$$

$$= \frac{223{\cdot}18}{1{\cdot}38} \times \sqrt{{\cdot}007} = 161 \times {\cdot}084 = 13{\cdot}52 \text{ feet per second.}$$

The difference, even allowing for the additional $\frac{1}{2}$ inch fall, is very marked, but the results are not really comparable, being calculated on widely divergent lines from dissimilar conditions.

A very complete and interesting example of sluicing on an extensive scale is shown by the plan in fig. 173, which refers to the Canada tidal basin at Liverpool.* The main culverts are constructed partly in masonry and partly in iron. Those of iron are circular in section and lined with a layer of Portland cement $\frac{3}{4}$ inch thick, which is secured by dovetailed ribs or keys at close intervals along the castings. This work, although completed twenty years ago, is still sound and intact, exhibiting no signs of erosion or decay.

The centre of the basin is brought within the scope of the discharge by outlets in the floor of the northern portion, which is laid with concrete. The sluicing pipes are arranged in radiating lines beneath the floor (fig. 172), each being provided with a series of upper outlets along its length, and terminating in a splayed opening. To protect these openings heavy frames or discs of greenheart (fig. 174) are laid over them as covers, being secured by four strong links to foundation anchorages. When the sluices are not in use, these discs lie at rest upon their respective outlets, but under the pressure of flowing water within the culvert they are raised to the full extent allowed by the links, and the water rushes out in the form of annular jets, sweeping the circular area within its range.

This arrangement has been found extremely effective for the purpose

* G. F. Lyster on "Dock Extensions at Liverpool," *Min. Proc. Inst. C.E.*, vol. c.

intended, but in view of the increase in depth continually demanded by modern shipping, a concrete floor to a basin is a feature which cannot be considered free from inconveniences. No deepening of the basin is possible without its removal, which must prove a costly and troublesome undertaking.

Sluicing is carried on daily at the Canada Basin, but the maximum effect is obtained at low water of spring tides, a time when the basin is very shallow, and when the inner docks can afford to part with a considerable amount of the water impounded on the flood tide. The water in the docks is always levelled with the incoming tide two hours before high water, within which period the operations of docking and undocking are carried on.

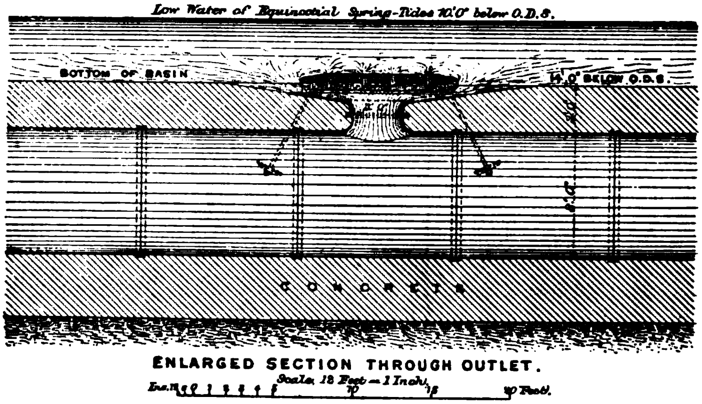


Fig. 174.

Sluicing on a large scale is a prominent feature of ports bordering on the English Channel. The method usually adopted is that of impounding a quantity of water during the flood tide, in a basin specially constructed for the purpose. At high water the sluice gates of this basin are closed, and the contents retained until a suitable period about low water, when the gates are opened again. The discharge of a large volume of water is found to be absolutely essential to the maintenance of entrance channels so subject to the introduction of sand by a littoral current, with its attendant deposition. The rate of discharge provided at Dunkirk and Calais is about 500 cubic yards per second, and the effective duration about three-quarters of an hour.

The recent improvement works at the port of Ostend (fig. 175) comprise a considerable enlargement of area in the sluicing enclosure there, concerning which M. Van der Schueren* makes the following observations:—

"Ships drawing much water will be able to enter the port by favour of

* Van der Schueren on "Travaux exécutés récemment et en cours d'exécution au port d'Ostende," *Int. Nav. Cong.*, Paris, 1900.

the tide, to reach the quay of the new outer harbour, and to remain there afloat; but, to this end, it is necessary to maintain a draught of 26 feet at low water.

"If it were considered essential to obtain this result by means of dredging, it is to be feared that the cost of the undertaking would be considerable, even excessive, and that the cumbersome appliances necessarily employed for its execution would be found only too often usurping, in front of the quay walls, berths destined for commercial vessels.

"Dredging, therefore, would constitute a drawback—a serious danger even—for navigation at the port; and the maintenance of great depths could with difficulty be assured by this means alone.

"In regard to ports on the Belgian littoral, the rapidity with which deposits of mud accumulate, in channels withdrawn from the action of natural or artificial currents, is well known.

"Under these circumstances, the utility of a sluicing basin would appear to be incontestible. The sluices are designed to supplement the action of the upper waters and of tidal currents, with a view to maintaining uninterruptedly, along the tidal quay of the new outer harbour, the assigned depth of 26 feet, without having recourse to continual dredging.

"The sill of the sluice is located 13 feet below datum, differing in this respect from existing sluices, the sills of which are level with, or not below, low water datum.

"The arrangement adopted is justified in respect of the efficacy of the current. Calculation, in accordance with observed results, enables it to be determined to what degree the useful work of the sluice is increased in this way.

"In his inquiry into the improvement of ports on a sandy beach the late M. Mey demonstrates, in effect, that in ordinary conditions, relative to the dimensions of the sluice and the reservoir basin, the useful effect of the effluent varies in the ratio of about 1 to 6·5 when the sill of the sluice, assumed primarily at the level zero (low water), is lowered afterwards to 13 feet below this datum."

The following are particulars of the sluicing arrangements at Ostend:—

Name of Basin.	Area.	Number of Sluices.	Width of Opening.	Level of Sill with reference to Local Zero.
	Acres.		Feet.	Feet.
Écluse Militaire, . .	29½	3	Two each 19½ One 39	– 5⅛
Écluse Française, . .	64	2	19½	+ ⅞
Écluse Léopold, . .	42⅓	6	13	+ 1¼
New Basin, . . .	192½	6	16½	– 13

To prevent the sluicing basin itself from being silted up, it is in some cases allowed to be filled only on the top of high water, when the influent is comparatively clear. This is the case at Honfleur. Elsewhere, as at

Ramsgate and Dover, the basin has been divided into two separate sections by a dividing bank, and one of these sections has occasionally been used to cleanse the other. Another expedient is to feed the reservoir with inland fresh water. In this connection, it is desirable to note that the specific gravity of fresh water being less than that of salt water, there is a marked tendency for fresh water to flow over the surface of the salt water, and it has been stated that the effect of scouring with the former does not extend to depths greater than 9 feet. *

At the port of Dublin a considerable area of strand of the estuary of the River Liffey is enclosed by a low retaining wall, which is submerged above half-tide level. When the tide falls below this level, the ebbing water converges to a contracted outlet, and produces a very effective scour at the mouth of the harbour.

It is very necessary to emphasise the danger of excavation in front of a discharging sluice. Even when a masonry apron of considerable width has been provided, the ground immediately beyond it has been found eroded to such an extent that measures have had to be taken to prevent serious damage. A hole, 6 feet deep, was formed at the edge of a stone apron, 80 feet in width, at the low-water basin, Birkenhead, and all attempts to fill up and reduce the hole by the discharge of large blocks of rubble stone into it were ineffectual. The same results were experienced at Dunkirk, where the sill of a former sluicing basin was found undermined to a depth of 13 feet.

Scraping and Scuttling.—This method consists in stirring up the deposit by mechanical means, to enable it to be carried away by an existing outward current. At Tilbury basin, harrows are employed for the purpose, aided by high-pressure water jets worked from a small tug during the ebbtide. The commotion caused by the revolving propeller itself of a tug with light draught in shallow water will also cause a very effective disturbance of mud. The same method with a larger vessel has been successfully employed for removing sandy bars at the mouths of rivers.

Dredging.—Dredging, as a means of channel maintenance, and distinct from deepening work, is open to the objection already stated, that it obstructs the navigable way. Having in view the soft nature of the material to be dealt with and the necessity for continuous removal of shallow deposits rather than the intermittent excavation of large accumulations, suction dredgers form the most useful type for maintenance work. Grab dredgers are also valuable in confined spaces, but the bucket dredger can only be usefully employed in large and unconfined areas, where a considerable bulk of material has to be excavated.

In the case of a suction dredger, the mud in the intake pipe forms a comparatively small percentage of its contents—averaging, say, from 30 to 40 per cent.—and of this a large proportion may be expected to pass out with the overflow water from the hopper into which it is discharged. The

* *Min. Proc. Inst. C.E.*, vol. lxvii., p. 461.

quantity of escaping material is capable, however, of being greatly reduced by a device due to Mr. A. G. Lyster and already referred to (p. 89).

We now pass on to a consideration of the structural features of locks and entrances.

Lock Foundations.—On the subject of foundations much that is stated in the chapter on Dock and Quay Walls is equally applicable in the present instance and need not be here repeated. There are, however, some contingencies and expedients peculiarly characteristic of lock construction which call for special notice and explanation.

The walls of locks differ from the ordinary type of dock walls in that they derive a considerable amount of support from the floor, especially if, as is usually the case, the latter has the form of an inverted arch or, if a flat floor, has curved haunches tangential to the side walls, or, failing that, is sufficiently thick to admit of the existence of a virtual arch within its limits.

The floor, on the other hand, without much assistance from hydrostatic pressure, has frequently to restrain the uplifting tendency induced by this lateral weight. The effect is more particularly felt in cases such as the lock at Bremerhaven (fig. 206), where there is no artificial floor, though in the instance cited the stress is minimised by the use of bearing piles beneath the walls.

As a general rule, hard rock and stiff clay, in which there are no springs, do not call *per se* for any artificial covering, except such as may be judged necessary to protect their surfaces from the softening and scouring action of water. On the other hand, alluvial deposit, sand, gravel, and other incohesive strata, need the confinement afforded by a superimposed mass in addition to the lateral support of sheet-piles. Earth of a porous nature, moreover, is not only unsuitable for a natural floor, but is equally undesirable as a foundation for an artificial floor, owing to its efficacy as a medium for the transmission of water pressure, on which account any covering laid upon it should be both strong and impervious.

The point of perhaps the greatest importance in connection with lock foundations is that of the treatment of boils or springs, such as are often encountered in works of this class. The type of foundation most likely to cause trouble in this respect is that in which a pervious stratum lies between two others of an impervious nature, the upper of which has been pierced or is fissured by a natural fault. The water-bearing stratum may then discharge copiously under considerable head, owing to a connection with some external supply located, often unsuspectedly, at some remote inland source. The following may be cited as an illustration germane to the point.

The site of the Albert Lock at Hull* consists of consecutive layers of silt, peat, boulder clay, sand, boulder clay, sand, and chalk. Soon after the lower bed of clay had been laid bare in the course of excavation there occurred numerous and powerful inbursts of brackish water charged with yellow sand. The source of the trouble was primarily attributed to the

* Hawkshaw on "The Albert Dock, Hull," *Min. Proc. Inst. C.E.*, vol. xli.

River Humber, but the fact that the sand between the two beds of clay was grey and loamy, whereas the water-borne sand was yellow, induced the engineers to make trial borings through the lower clay. This was found to be a stiff brown layer, 42 feet in thickness, and the borehole remained quite dry until the bottom was reached, when water charged with yellow sand flowed up the hole with considerable force, showing that the boils had their origin in the sand bed which immediately overlay the chalk. As the chalk wolds extend over a large area, attaining an elevation of 500 feet at no great distance from Hull, and giving rise to copious springs at their base, it was then considered probable that the influent was mainly due to land water accumulated in the chalk, though the fact that the stream was brackish indicated some connection with the sand beds of the River Humber.

Sometimes the source of leakage, being nearer at hand, is more obvious. In the reconstruction of the Canada Lock at Liverpool, the site of which comprises an alluvial bed overlying two layers of boulder clay, intersected by a bed of sand and gravel of varying thickness, considerable difficulty was experienced at first owing to inbursts from the river through the sand. Excavations at the time were in progress, continuously within the lock chamber, under protection of the gates, and intermittently at the outer sill, at low water of spring tides. The removal of the upper clay in both situations was coincident with the flooding of the lock chamber at high tide, clearly under the head afforded by the water in the river. It was found impossible to keep down the water in the lock, and the interior work had to remain in abeyance until the outer sill was completed.

The larger area involved in the construction of locks and entrances generally renders it difficult, and not always advisable, to adopt the method of treatment recommended for infiltrations of water to wall foundations—viz., to lead them to some suitable spot where they can be provided with a vent. Discharge within the lock itself is inconvenient in the case of small streams and impracticable in the case of large ones. On the other hand, to convey a discharge outside the lock area would be a matter often attended by needless difficulty and expense. Furthermore, there is the risk that the effluent might carry with it material in suspension, unless it were entirely checked by a counteracting head.

In view of the diversity of conditions under which constructive operations have to be carried on, it would be obviously impossible to lay down any general rules of procedure in case of leakage arising from boils or springs. All that is permissible is to briefly indicate a few of the methods which have been successfully adopted in actual cases, putting on one side altogether the question of their applicability elsewhere.

1. Where the discharge has been slight and of the nature of an infiltration, it has been easily checked by the rapid deposit of a large bulk of concrete upon the spot, the concrete being mixed fairly dry, so as to allow for its admixture with the water *in situ*.

2. Where the discharge has been greater, but sufficiently moderate not to interfere with work in the vicinity, it has been allowed to find its way into the pumping well in the ordinary manner (*i.e.*, by open "grip" or drain pipe), and any sand, which accompanied it, filtered by laying straw, bags, strips of canvas, &c., over the source and weighting them down.

3. Where the discharge has been more rapid still, proceeding from a definite "blow," with a tendency to diffusion, it has been concentrated into a special iron pipe which led temporarily to a pumping well, or in another case was carried up to a height equal to the head of discharge. There is a danger, however, attached to this method of repressing the flow by a counteracting head. The general pressure is in no way relieved, and there is every likelihood of the blow re-asserting itself at another weak spot, so that the horizontal duct is a preferable course to adopt. At a later period, the pipes referred to were grouted with cement under pressure, and in due time, after the cement had set, the projecting portions were cut off. The connection between the pipe and the blowhole necessitated careful and ample packing with rubble and clay or cement in bags, so as to secure a thoroughly watertight joint. In one instance a hollow hemispherical casting was employed to collect the outflow. It was 3 feet in diameter, with an upper flanged connection for a 6-inch pipe, and sat upon a ledge surrounding the hole, below the foundation level, in which position it was concreted.

4. Where the discharge has been so great as threaten to overpower the pumps, it has been deemed advisable to block the holes, temporarily, with rubble and clay puddle, tipped in large quantities, until the affected area could be isolated by an enclosure of whole timber sheet piling. One hole, treated in this way, is recorded as having absorbed several hundred cubic yards of puddle.

5. Finally, where the pumps have actually been overpowered, the water has been allowed to rise to its natural head and the excavation completed by dredging. The foundation was then piled and the pile-heads cut off by divers to one uniform level. A covering of jute cloth was next laid over them and fastened there while concrete was deposited upon the site by skips opening at the bottom. When the concrete had reached a height sufficient to counteract the under-pressure, the area was pumped out again and the work resumed in the open.

One essential feature stands out prominently as the outcome of much experience—the necessity for adequate pumping power. It is, in fact, wiser to provide it in excess of all anticipated requirements, rather than to run the risk of a stoppage of the works at some critical and momentous period. At the same time, it must be borne in mind that there is a judicious limit to be observed. If the water be continuously and heavily charged with silt or sand, which cannot be checked, it is evident that a void is being formed somewhere, and that settlement of the foundation will be the

probable result. Under such circumstances a suspension of pumping operations becomes imperative.

Occasionally, leaks have been found to develop in the floor or sills of a lock or entrance subsequent to the completion of the work. In such cases the *locale* of the leak has been bored through to the underlying stratum and stand pipes, fitted into the holes, have been filled with cement grout, from a considerable height, to be cut off later as already described. This operation is best carried out at a time when the pressure of water within and without the lock is the same. Provided the holes are sufficiently close together, the whole of the underside of the floor may be coated in this way with a thin watertight diaphragm. Fissures in rock can be treated by the same process, and it is a common method for grouting up the interior of a cast-iron roller path after it has been adjusted by wedges and holding-down bolts to its proper level on the gate platform.

Another course of treatment for cracks and fissures is that called *stock-ramming*, and consists in inserting into the borehole pipe lumps of clay worked up with cement or hydraulic lime, sand mixed with iron filings and sal ammoniac (rust cement) or stiff concrete, the material being forced home by blows from a heavy ram worked by hand or steam-power.

Open joints may be caulked by rolls of canvas, partially filled with soft cement. Large fissures are sometimes cut out, so as to form a rectangular recess into which a block of masonry is fitted, wedged up, and grouted.

Cracks will often occur near the centre of a lock floor, owing to the unequal distribution of pressure over the foundation area, arising from the greater weight of the side walls. These manifestations of weakness may be prevented by adopting a floor, the section of which constitutes an actual or virtual inverted arch.

The problem of the proper distribution of pressure over a lock area is a very important one, particularly if the strata be irregular and water-bearing. A variety of methods have been exemplified in different localities.

If the ground be of an uncertain or treacherous character, such as clay interspersed with pot holes of quicksands, it will be well to effect the uniform distribution of the superimposed weight by the interposition of timber planking laid horizontally and arranged so as to break joint.

A loose sandy foundation may be somewhat consolidated by driving a series of short piles at close intervals. A row of external sheet piling should not be neglected.

An ingenious method has been devised for transforming a sand or gravel foundation into one of concrete, by impregnating it with Portland cement under air pressure. The following details relate to the manner in which the operation was carried out at the Port of Vegesack, near Bremen, on the River Weser:—*

A pipe or shaft, $1\frac{1}{2}$ inches in diameter, pointed at its lower end and per-

* Neukerch on "Constructing Foundations by forcing Cement into Loose Sand and Gravel by Air," *Min. Proc. Am. Soc. C.E.*, vol. xxx., p. 284.

forated there with three or four holes of $\frac{3}{8}$ inch diameter, was sunk under compressed air into the sand until it reached a depth varying from 16 to 19 feet. In the air pressure supply pipe provision was made, by means of suitable branches and stopcocks, for connecting therewith an apparatus which, with the aid of an injector device, enabled any desired quantity of cement powder to be fed into the air current. While this was being done, the pipe was slowly withdrawn in an upward direction, so that the cement was thoroughly diffused throughout the bed, which was full of natural moisture. The cement was supplied dry and warm air was used. Consecutive areas, from 8 to 12 inches square, were treated in this way, and the concrete allowed sufficient time to set before being built upon.

Principal Constructive Features.—Apart from the question of the floor and its foundations, the following (illustrated in fig. 176) are the most important features in the construction of entrances and passages generally:—(1) the sills, (2) the platforms, (3) the side recesses and chambers, (4) the walls, and (5) the levelling culverts. The subject of the means adopted for closing the entrance is reserved for an independent chapter.

1. *Sills.*—If for caissons, these will constitute straight lines in plan, normal to the axis of the waterway; if for gates, each will consist of two straight or curved lines intersecting at the centre. The level of the sill will generally be somewhat higher than the floor of the chamber in order to avoid sinking the gate or caisson platform below the floor level. This, however, is often done, more especially in the case of caisson platforms, which are not so extensive

Fig. 176.—Elevation of a Dock Entrance.

OUTER WATERWAY.

as gate platforms. The objection is the great tendency for any depression in the floor to form a mud trap, but this may be partially obviated by arranging the culvert inlets so as to exercise their influence at such parts. The sectional profile of a sill is often curvilinear, but the outlines of modern naval architecture render it desirable that the sill should be as flat as possible. The height of the sill depends upon the amount of cover required to form a watertight joint with the gate or caisson, and the clearance necessary for truck-wheels, rollers, or slides, as the case may be. Six or eight inches will generally be sufficient in the first case, and the total depth usually varies from 18 inches to 3 feet. The vertical abutment face of the sill may be formed by stone, wood, or iron, assuming that there is always a wooden member of the gate or caisson to come into contact with it. The dressing of this timber face necessitates great care and good workmanship, for upon a close-fitting joint depends the absence of leakage.

On account of their proximity to the unprotected earthen floor of a dock, the sills of passages and the inner sills of locks are at times subject to very great hydrostatic pressure, if the underlying stratum be in any degree porous. Instances have even occurred in which, with a rock foundation, water has percolated into the bed joint between the sill and the rock, causing the former to uplift. To minimise the danger arising from this cause it will be advisable to pierce the sills with a series of vent holes, lightly covered with pieces of flagstone. If the bed joint remain intact these vents will not be called into action, but if through any mischance water should penetrate beneath the sill at a time when there is little or no hydrostatic counteraction, it is infinitely preferable that there should be a means of escape for the water rather than that the full effect of the fluid pressure should be exerted against the underside of the sill to its detriment and possible disruption. From the foregoing considerations it is obvious that weight and homogeneity are distinct advantages to a sill.

To prevent undermining by the wash of the tide or the scour of a current, the outer sills of entrances should be provided with a masonry or concrete apron extending some distance in front of the sills.

2. *Platforms.*—These form the floor over which gates and caissons are moved in and out of position. If for gates fitted with truck wheels or caissons with rollers or slides, they will be provided with granite, or iron, or steel tracks, the last two firmly bolted down to the masonry or concrete. Metal roller paths for gates form segments of circles in plan, and their upper surfaces are bevelled to the inclination of the truck wheels, which are truncated cones, on account of the greater amount of travel to be performed by the outer edge. The axis of the cone will intersect the axis of the pivot. Caisson tracks are either flat metal surfaces or rails. Occasionally, the wheels are attached to the floor, and the track or sliding surface to the underside of the caisson. A platform should be sufficiently strong

to support without settlement any weight which may be concentrated on a limited portion of its area. The excess weight of a large greenheart gate, over and above its flotation, may amount to as much as 50 tons, and this has to be divided between the pivot and, say, two truck wheels, so that the three points of contact are undergoing a stress equivalent to a pressure of nearly 600 feet of water more than the remainder of the platform area. The disparity in pressure will be greatly accentuated for intermediate and outer gates at such times as when the lock happens to be dry; and as caissons are frequently utilised as avenues for traffic, it is well to remember that the effect of any dead or moving load which they carry is transmitted direct to the platform below. The bedding and adjustment of the wheel tracks is then a matter for careful attention.

3. *Side recesses* for gates are usually curved in form and sufficiently deep to admit of the gate receding well beyond the face line of the side walls, in order to avoid concussion with passing vessels. A gate recess terminates in two returns, or quoins, called from their shape the *hollow quoin* and the *square quoin* respectively. The former receives the heelpost of the gate

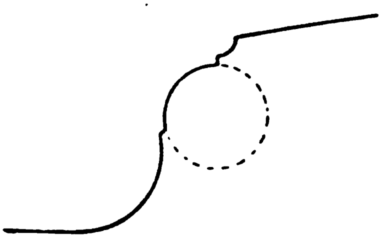


Fig. 177.

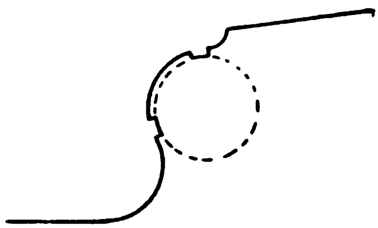


Fig. 178.

and, accordingly, is concave in plan, forming a circular segment. Combined with its curved junction with the side wall it may be described as a modified Ogee or Cyma Recta. There are two types of hollow quoin. One, which finds favour in this country, provides a cylindrical surface in close contact with the heelpost for a considerable portion of its circumference. This design (fig. 177) entails very accurate and careful dressing, and is attended by the inevitable wear of the contiguous surfaces, resulting in leakage, though not to the extent which might be supposed. The alternative plan (fig. 178), in vogue in Holland, is to limit the amount of watertight contact to a narrow straight face, about 8 inches in width, the dressing and polishing of which, being a plane surface, is accomplished with greater facility than that of a cylindrical quoin. At the outer edge of the quoin there is another close-fitting strip to prevent the passage of small floating objects. In both forms of quoin the friction of movement may be diminished by affording a slight play in the pivot, by which the gates revolve out of contact with the quoin. Hydrostatic pressure causes the surfaces to resume their watertight abutment. The joints of hollow quoins are preferably bedded in lead for a depth of 6 inches from the face. The

stone should be of a very hard and durable quality. Granite is almost invariably used, but greenheart timber has also been employed with, it is stated, most satisfactory results. The wear of the heelpost is said to be less, and the cost of dressing the surface of the quoin much reduced.*

At the base of the hollow quoin is situated the foundation stone to receive the gate-pivot casting.

In cases where chains are used for manœuvring the gates, it will be advisable to attach a check chain from the top of the mitre post to a volute or other spring fixed in the neighbourhood of the square quoin, to avoid violent impact against the sill.

Recesses for sliding and rolling caissons (fig. 179) are usually rectangular chambers constructed normally to the axis of the passage. They require to

Fig. 179.—Caisson Recess at Greenock.

be slightly longer than the width of waterway, and to be slightly wider than the caisson itself. In some cases sufficient room is left between the caisson and the side wall of the chamber to allow of men conveniently effecting repairs to the caisson should such be necessary, the chamber being rendered watertight, temporarily, by timber dams. A strong covering is expedient on account of the traffic overhead.

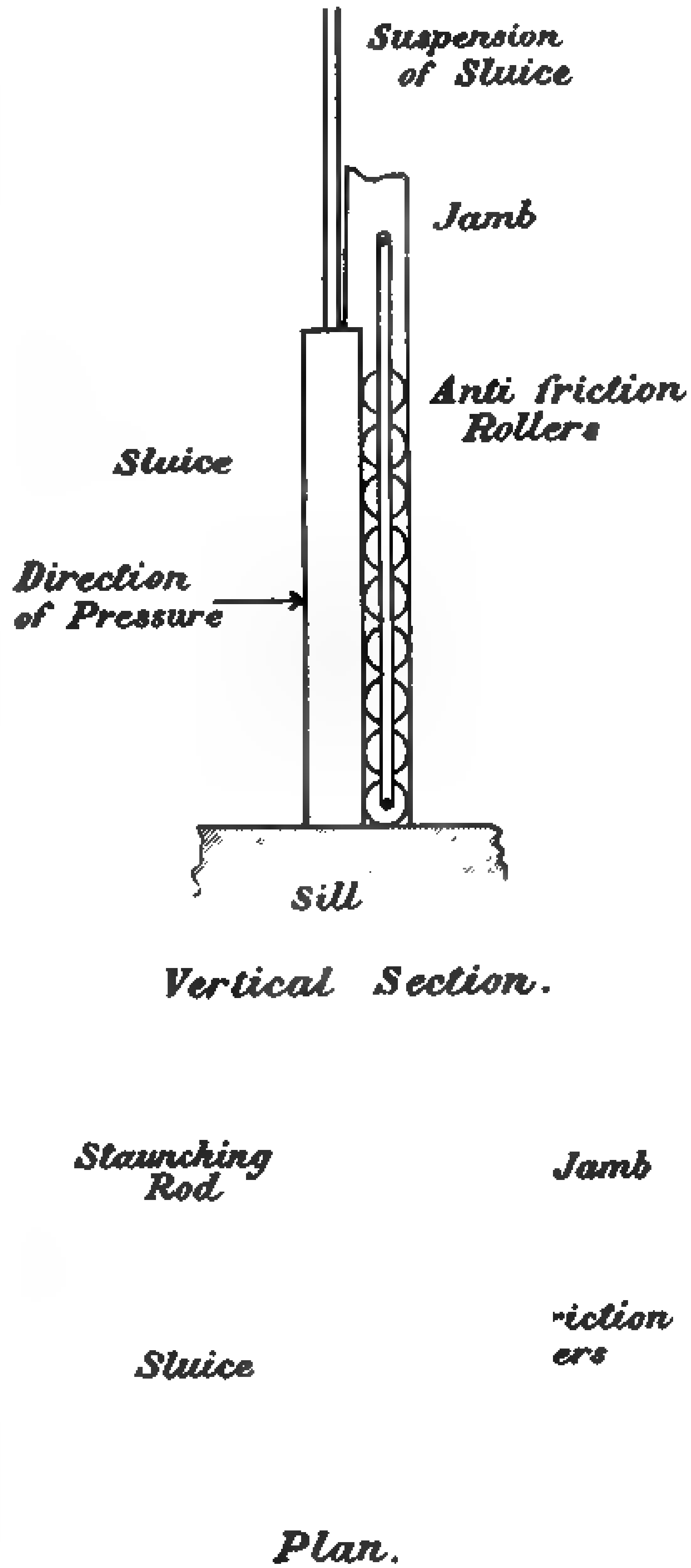
4. The *side walls* of a lock are preferably constructed without any batter on the face. With the water at widely varying levels there would be a danger of two vessels, locking outwards side by side, nipping each other unless the walls were plumb. Where a ship caisson is used for closing the entrance, and, for that purpose, is floated into grooves in the walls, a slight batter is inevitable, but the method is unusual for locks and the contingency remote.

* Moncrieff on "Dock Gates of Iron and Steel," *Min. Proc. Inst. C.E.*, vol. cxvii.

5. The *levelling culverts* may with advantage be arranged so that their inlets are behind the hollow quoin and on a level with the gate platform. In this way they assist to keep the platform and wheel tracks clear of mud. Where caissons are employed, the culverts may have their openings into the caisson chamber with the same object in view.

The flow of water through culverts is regulated in several ways, one or two of which will be briefly described.

(*a*) At a certain point, usually near the inlet, the culvert is intersected by a *clough-paddle* or *penstock* (fig. 223). This consists of a substantial frame of wood or iron, faced with a plane surface sliding in side grooves, and having horizontal bearings against a head and sill in the roof and floor of the culvert respectively. A vertical shaft above the culvert permits the paddle to be entirely withdrawn from the sectional opening of the culvert. Raising and lowering are performed by manual labour or by hydraulic or other power. When the culvert is not in use the paddle is kept down. By lifting it communication between the outer and inner water is established, and if there be any difference of level a current is immediately formed. Ordinary cloughs are provided with stone (generally granite) jambs, head, and sills, the sliding surfaces being polished.



Figs. 180 and 181.—Stoney Sluice.

The paddle is slightly larger than the opening—about 6 to 12 inches each way—and may be either tapering in thickness or with parallel faces. It is a judicious arrangement to have duplicate paddles, one being actuated by hand in case of mishap to the other worked by machinery.

(β) *Stoney sluices*, so-called from the name of their inventor, have the friction of the bearing surfaces during movement very much reduced by the employment of rollers. The doors are of steel, and a watertight joint is formed by the engagement of a rod in a V-shaped groove. Figs. 180 and 181 explain the arrangements adopted.

(γ) *Fan doors* (*portes en éventail*) are adopted in some instances abroad. They are in the shape of a right-angled triangle in plan (fig. 182), with a vertical axis at the corner, formed by the intersection of two plane surfaces of unequal area. When in position the smaller wing, bearing against a wood-lined frame, cuts off the culvert connection. To open the gate the larger wing has to revolve within a cylindrical chamber. A small discharge

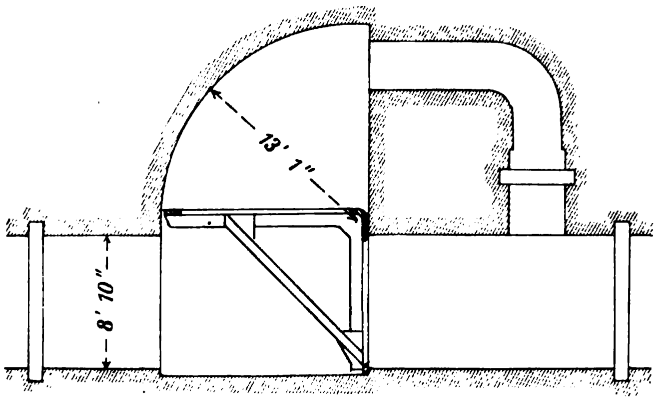


Fig. 182.—Plan of Fan Door at Dunkirk.

pipe fitted with a valve serves to set the gate in motion. While the valve remains closed the up-stream pressure keeps the gate shut. As soon, however, as the valve is opened the water in the cylindrical chamber escapes, down-stream pressure is introduced into the chamber, and the difference causes the gate to revolve on its pivot, in virtue of the unequal areas exposed.

(δ) Other doors or gates are in one plane surface throughout, turning upon a vertical axis slightly out of centre. By opening a small valve in the wider panel the pressure on that panel is reduced below the pressure on the other panel, and the gate revolves so as to set itself in a line with the stream. Closing the valve and giving the gate a slight sideways displacement causes the current to act with greater effect on the larger surface, so that the gate automatically swings to. It is locked in position by a turn of the wooden side post.

Duration of Levelling Operations.—It is often desirable to know how long it will take to level up a lock from a lower to a higher level through the medium of a culvert. If the source from which the water for the purpose is drawn be maintained at a constant level, or so nearly constant as to be conceivably treated as such, the calculation is a simple one. The theoretical velocity is $v = 8\sqrt{h}$, as previously explained. This multiplied by the sectional area of the culvert, or of the culverts if there be more than one, combined with a suitable coefficient of discharge, gives the quantity of water passing in unit time, whence the total time is obtained by dividing into the quantity of water required to fill the lock. Therefore, algebraically, the time in seconds,

$$t = \frac{Q}{8\,a\,c\sqrt{h}}, \quad . \quad . \quad . \quad . \quad . \quad (42)$$

where Q is the quantity required in cubic feet, a the culvert area in square feet, and c the coefficient of discharge—varying from ·5 to ·6, according as the culvert is long or short.

If the source of supply be not maintained at a sensibly constant level during the process of filling, as when two docks, whose areas are not very excessively unequal, have to be brought to a common level by intercommunication, a suitable formula may be deduced from the same principles, as follows:—

In addition to the previous notation, let A_1 and A_2 represent the areas of the docks in question, h_1 the height by which the lower dock (A_1) is raised, and h_2 that by which the higher dock (A_2) is lowered. Then $h_1 + h_2 = h$.

The initial velocity of influx is $8\sqrt{h}$, the final velocity is zero; the mean velocity, therefore, is $4\sqrt{h}$. The rate of influx thus becomes $4\,a\,c\sqrt{h}$.

The quantity of water required to be transferred is, indifferently, $A_1\,h_1$ or $A_2\,h_2$—that is,

$$A_1\,h_1 = A_2\,h_2;$$

but

$$h_2 = h - h_1.$$

Therefore

$$A_1\,h_1 = A_2\,(h - h_1),$$

or

$$h_1\,(A_1 + A_2) = A_2\,h.$$

That is,

$$h_1 = \frac{A_2}{A_1 + A_2}\,h.$$

Substituting this value for h_1 in $A_1\,h_1$, the quantity of water required to be transferred (Q), and completing the equation as in the previous example (42), we finally obtain—

$$t = \frac{A_1\,A_2}{A_1 + A_2} \times \frac{\sqrt{h}}{4\,a\,c}. \quad . \quad . \quad . \quad . \quad . \quad (43)$$

Throughout the remarks which have been made in connection with structural operations it has been found convenient to use the word Lock as a more or less generic term to include Entrance and Passage as well.

Unless the sense absolutely precludes such an interpretation the reader will consider the principles laid down as applicable and common to all forms of narrow dock waterways. In one respect alone does a passage materially differ in design from a lock. A lock provided with gates has them all (with the possible exception of storm gates) pointing in the same direction, whereas, in a passage, the gates point in opposite directions in order to exclude water from either of the docks which it serves to connect.

Having commented as fully as is practicable within the limits imposed by restrictions of space, upon the various matters appertaining to the design and construction of locks, we now pass on to a brief review of some prominent examples selected from harbours in various parts of the world.

Canada Lock, Liverpool.

Constructed in 1857, with a single chamber, having an effective length of 498 feet, a width of 100 feet, a depth of 35 feet 9 inches below coping, and a draught of 26 feet 9 inches on sill at H.W.O.S.T., this lock was deepened in 1895 to a draught of 33 feet on sill, lengthened to 602 feet, and divided by a pair of intermediate gates into two chambers of 200 and 402 feet respectively. In addition to the three pairs of gates, the lock pierheads are fitted for the reception of ship caissons in the event of repairs being necessary to the outer sills.

Fig. 183.—Section of Old Canada Lock, Liverpool.

The old lock was constructed entirely in masonry and intended to serve the additional purpose of a graving dock. Hence the peculiar form of section adopted and shown in fig. 183. The recessed panels in the side walls were for the abutments of shores to the sides of vessels. In the course of alteration these panels were filled up, as also were the lower sluicing culverts, except for short lengths on each side of the gates, where they are now utilised as levelling culverts.

The improvement work of 1895 consisted in removing the old masonry floor and replacing it by one of concrete, at a depth of 3 feet 3 inches lower than the new sill level, founded on the boulder clay which underlies the whole site. The concrete was composed of 8 parts of gravel to 1 of Portland

cement, with a large proportion of sandstone and granite burrs thrown in. The thickness of the new floor averages 7 feet, and the upper surface is coated with a 6-inch layer of granolithic concrete. A transverse section (fig. 184) shows the floor to be flat for a width of 80 feet and connected with the sides by circular curves of 10 feet radius. The side walls were underpinned with concrete in bays of from 12 to 15 feet in length. A gas- and water-pipe culvert, 5 feet in diameter, is arranged below the floor level.

The stone work comprises copings, hollow quoins, culvert quoins, caisson quoins, gate sills, caisson sills, culvert sills and heads—all of Scotch granite, with square quoins of sandstone.

The work was carried out in the following manner:—The outer sill in the tidal basin was reconstructed during low water of spring tides in small sections, within a piled dam, which was pumped out on each occasion. On the completion of the work a stank of concrete blocks was built across it

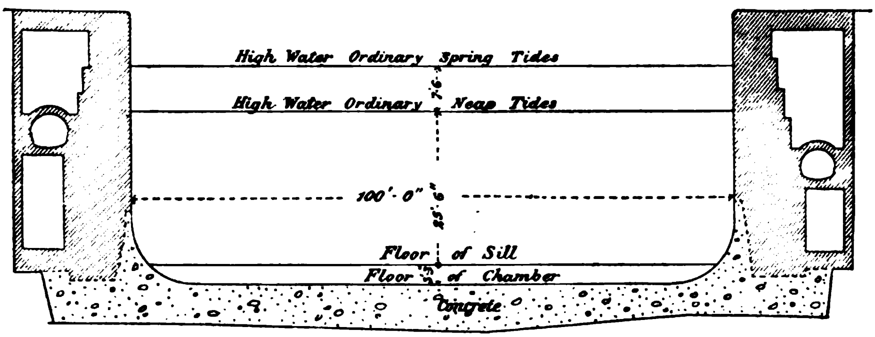


Fig. 184.—Section of Canada Lock, Liverpool, as deepened.

between the side walls of the lock, and carried up above the level of high water. These blocks were of uniform size, 11 feet 3 inches by 3 feet by 3 feet, each containing about 100 cubic feet. They were made in wooden moulds at least a fortnight before using, and were deposited by means of overhead steam travellers, double tracks for which, 64 feet wide, ran the whole length of the lock. To ensure watertightness, the blocks were bedded in cement mortar. At the same time, to facilitate their later removal, a sheet of common brown paper was interposed between the block and the mortar. The plan answered admirably, the blocks being perfectly bedded without the undesired adhesion. It is needless to add that the stability of the dam in no way depended upon the tenacity of the joints.

The inner end of the lock was enclosed by a cofferdam, constructed of piles and timber framing and filled with clay puddle. A section of the dam is illustrated in fig. 66 (p. 107). When the dams were completed no difficulty was experienced in bringing the work to a rapid and successful conclusion. Three chain pumps with wooden blades, 2 feet 6 inches by

6 inches, running alternately and intermittently, were found adequate to deal with all infiltrations of water.

The North Lock at Dunkirk.*

Prior to 1896 the port of Dunkirk was served by three entrance locks, the largest of which, the west lock, had a serviceable length of only 384 feet and a width of 69 feet. As far back as 1883 this accommodation was found to be insufficient, and in 1887 the project of a large new lock (fig. 185) was approved, at an estimated cost of 9½ million francs. The dimensions decided upon were: a width of 82 feet and lengths of 687 feet over all, 580 feet between outer sills and 558 feet available for actual use. The level of the sills was fixed at 16 feet 6 inches below the local datum (zero of marine charts), so that there is an available draught of 30 feet at lowest neap tides, 32 feet 6 inches at mean neaps, and 35 feet 9 inches at mean springs. The works were completed and the lock opened for traffic in 1896.

The lock is provided with three pairs of metal ebb gates, by means of which it can be divided into two chambers, with lengths of 351 and 229 feet respectively, for the purpose of reducing the period of locking for vessels of moderate or short length. The outer gates are furnished with strut frames as a support against rough seas.

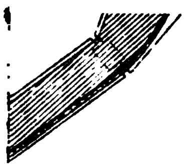
The filling and emptying of the lock are achieved by means of two longitudinal culverts of 11 feet 6 inches by 5 feet 9 inches sectional opening running, one on each side, from one end of the lock to the other. These culverts are closed at the extremities and near the middle by swing gates, of the type called *fan* gates (*portes en éventail*), and they are in permanent connection with the lock chamber by means of 16 transverse openings, each 6 feet 6 inches wide. The dimensions given to the culverts are such that the lock can be filled in six minutes under a head of 10 feet.

Ship caissons can be berthed at both ends of the lock in case of accidents and repairs. The opposite quays are in communication by means of a centrally situated two-leaved swing-bridge, with a single cart track, including a line of rails. A metallic culvert of circular section, 6 feet 9 inches in diameter, forms a syphon under the floor for the transmission of water, gas, electric, and hydraulic supply mains.

The area of 10 acres which formed the site of the lock between the outer channel and the inner docks was enclosed by means of two cofferdams, one at each end.

The outer dam (fig. 186) was based on the sill of an old sluicing lock after the removal of the masonry, closing the opening between the side walls. It consisted of a bank of sand having its outer slope covered by a thick layer of stiff earth (*épaisse couche de terre forte*), with stone pitching superadded as a protection against wave action. The inner dam formed a semicircle in plan, projecting into the adjoining basin. As in the previous

* Vide *L'Ecluse Nord et ses Abords*, Dunkirk, 1896.



case, it consisted of fine clean sand filling, carefully watered and rammed in thin layers.

The foundations of a neighbouring lock rest directly upon a very thick bed of fine sand which underlies the district, and a similar mode of foundation was contemplated in the first instance for the new lock. But the work also occupies a portion of the site of the old sluicing basin, and, on examination, it was found that very extensive excavation had resulted from water scour in front of the sluice gates, and that the sand bed had been disturbed to a considerable depth. Consequently, as it was desirable that so important an undertaking should rest upon a homogeneous base, it was decided to carry out a general scheme of close piling.

The piles employed were of oak, of 10 inches mean diameter, 14 feet 9 inches long under the floor of chamber, 16 feet 3 inches long under the sills, and 18 feet long under the aprons. The piles were pitched at distances proportionate to the thickness of the masonry, which attains 62 feet in the side walls of the pierheads and is reduced to 13 feet within the chamber. The number of piles was 6,300, and they were driven by ten steam-piling machines and three ringing machines.

The floor, which varies in thickness from 13 to 18 feet, is formed by a layer of brickwork,

Fig. 186.—Dam at Dunkirk.

set upon a concrete bed and covered by ashlar masonry (*moellons d'appareil*). The concrete, 6 feet 6 inches thick in the floor of the lock, 10 feet thick in the gate platforms, and 12 feet thick in the aprons, was composed of equal parts of hydraulic lime mortar (Tournai lime, trass, and sand), pebbles (*galets*), and broken material (*briques roches concassées*). The sectional profile of the floor (fig. 187) exhibits a flat centre, 42 feet 6 inches in extent, flanked by curves which are tangential to the side walls.

Fig. 187.—North Lock, Dunkirk—Section.

The side walls were executed generally in local brick and limestone, set in Portland cement mortar, with a facing of ashlar masonry. Normandy or Brittany granite was used for the sills, hollow quoins, caisson quoins, copings, square quoins, culvert apertures, and for the rounds of the pierheads above low water. The mortar was composed of 1 to 1½ parts of Portland cement to 1 of sand.

The North Lock at Buenos Ayres.*

It was at first proposed to lay out the northern entrance to the Madero Docks in a north-easterly direction from the north basin to the outer roads, where there is a long stretch of water having an average depth of 20 feet 3 inches below low-water level, and thence in a S.E. direction to the bar anchorage. This line, however, was abandoned as likely to involve an increase in silting, owing to its directly transverse situation in regard to the stream, and it was eventually decided to turn the channel as quickly as possible into the run of the river (see fig. 8, p. 37).

The north lock is 82 feet wide at the coping, has a length of 508 feet 6 inches between sills, and a draught of 22 feet over sills at low water. It is traversed by a swing bridge. A main service subway, 9 feet 10 inches by 7 feet 6 inches, in rubble masonry, lined with brickwork, passes under the floor. The general disposition of the lock will be readily grasped from an

* Dobson on "Buenos Ayres Harbour Works," *Min. Proc. Inst. C.E.*, vol. cxxxviii.

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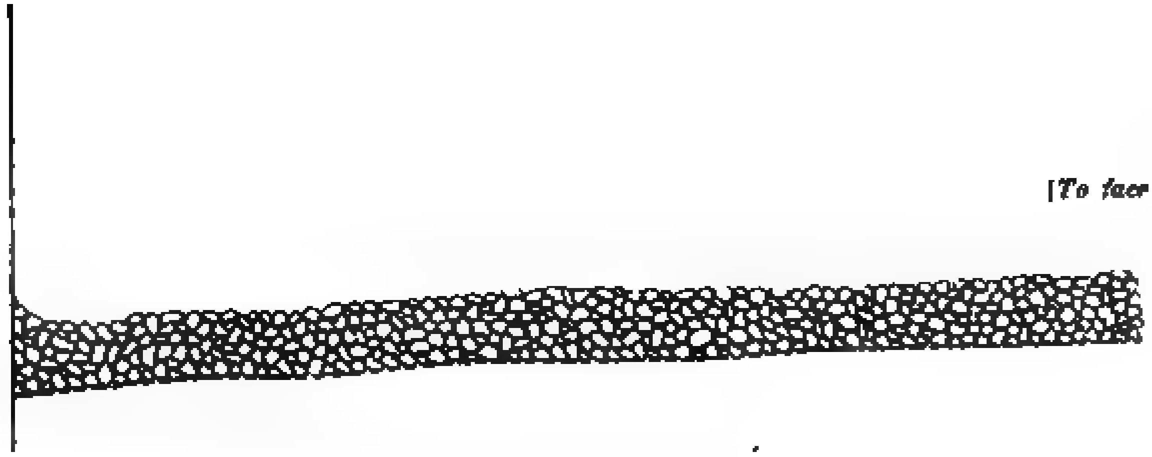

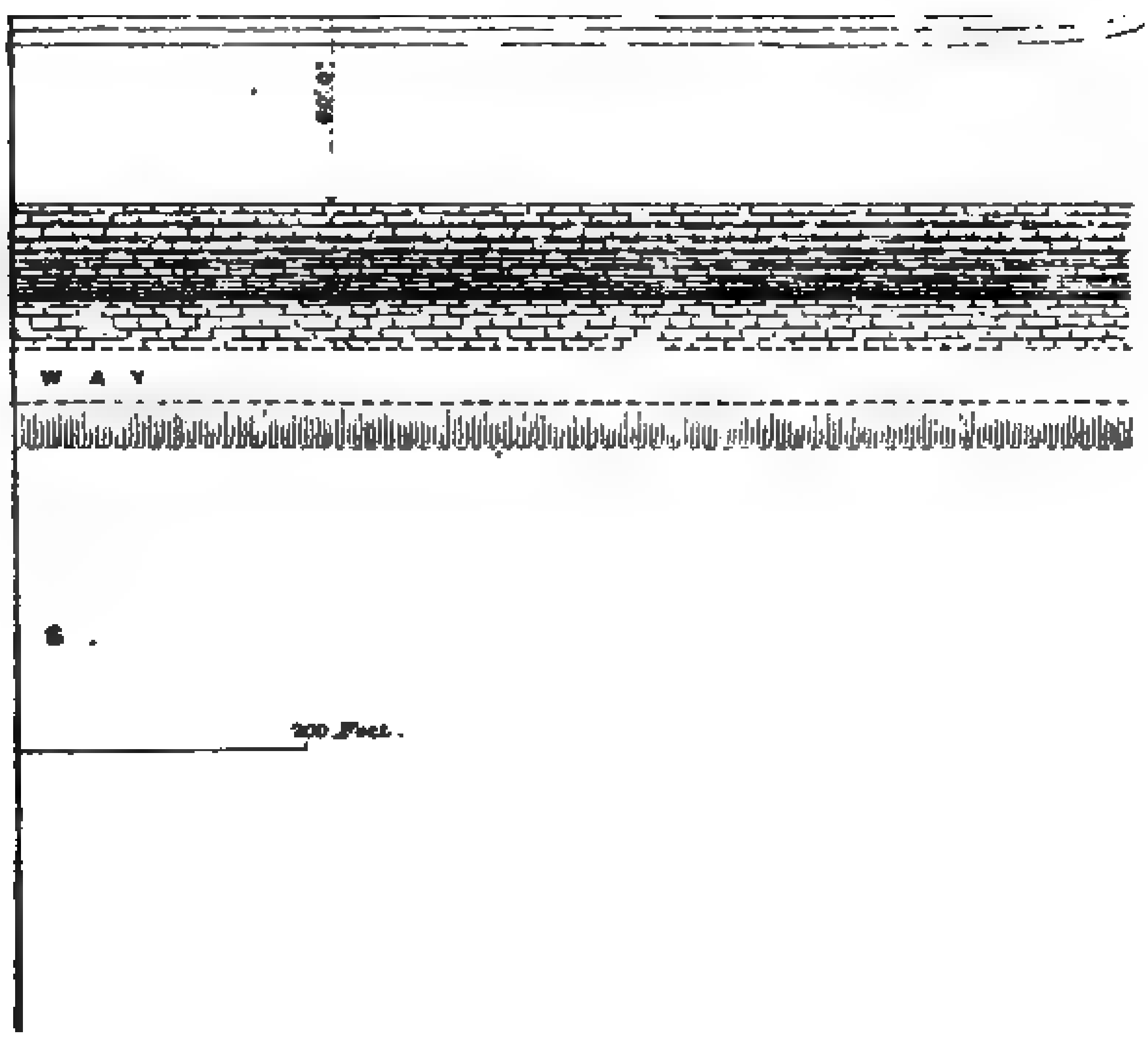

inspection of figs. 188 to 192. The following interesting experience occurred during its construction :—

A very large bed of running sand was met with just at the intended level of the bottom of the foundation of the north sill. The sand was so troublesome that all pumping had to be at once suspended, and the level of the bottom of the foundations raised and widened out so as to reduce the weight per unit area on the soft white *tosca* overlying the running sand. To overcome the difficulty an iron cylinder, 8 feet in diameter (fig. 193), was sunk through both strata into the hard *tosca* below, the excavation being performed by a diver. When the cylinder was well down, a good layer of strong concrete was put in, making the cylinder quite watertight below, while it was allowed to receive by lateral holes the drainage from the upper white *tosca*, at a level between the bottom of the foundations and the top of the running sand. A centrifugal pump, working continuously, kept the water in the cylinder below foundation level. Before building the masonry of the north sill, the entire surface was covered with a layer of concrete, 25 inches thick. Laying the concrete in bags, which was the method first attempted, did not succeed, and canvas in long strips was substituted, with the joints so placed that the water would run underneath. This plan answered well, and although the level at which the canvas was laid was only 2 feet above the running sand, the whole of the concrete was put in quite dry. When the invert was completed, the cylinder was filled with concrete and built over.

Eastham Entrance Locks.

Manchester Ship Canal.—There are three entrance locks (fig. 194) constructed in parallel lines pointing down the River Mersey, 600 feet by 80 feet, 350 feet by 50 feet, and 150 feet by 30 feet, respectively. The lower sill of the largest lock is 42 feet below high water of ordinary spring tides. This lock has culverts on each side, 12 feet high by 6 feet wide, which enable it to be filled or lowered, so that a vessel can pass through in less than ten minutes. Two 20-foot Stoney sluices adjoin the locks and assist to fill and lower the canal at tide and flood times respectively.

Entrances at Kidderpur Docks, Calcutta.

Plans and sections ef the entrance locks and passages at these docks which have already been referred to (p. 236) are shown in figs. 195 to 200.

Entrances at Barry Docks.

The harbour is approached by a sheltered channel, 450 yards long, enclosed by two breakwaters, the heads of which are 350 feet apart. There are two entrances—one, leading to a basin, is available for $2\frac{1}{2}$ hours before and $2\frac{1}{2}$ hours after high water; the other, known as the Lady Windsor

Lock, can be used at any state of the tide, having a depth of 16 feet of water at low water of ordinary spring tides. The basin entrance and the passage between the basin and No. 1 Dock are each 80 feet wide. The sills are curved, with a versed sine of 3 feet, and a central draught of 40·7 feet at high water ordinary springs, and 32·3 feet at high water ordinary neaps. Timber guiding jetties, 200 feet in length, are erected seaward of the basin entrance, and a masonry jetty, with timber fenders, 600 feet long, leads to the Lady Windsor Lock. This last has a length of 647 feet, a depth of 60 feet and a width of 65 feet. It is divided into two compartments by an intermediate pair of gates. The depth at the centre of the curved sills is 52·8 feet at high water of ordinary springs, and 44·4 feet at high water of ordinary neaps.

Eglinton Dock Entrance, Ardrossan.*

The walls of the entrance (fig. 201) were founded on rock excavated 4½ feet below the sill, which is level with the bottom of the dock and tidal basin; the gate floor is 18 inches lower than the sill. The sluices on

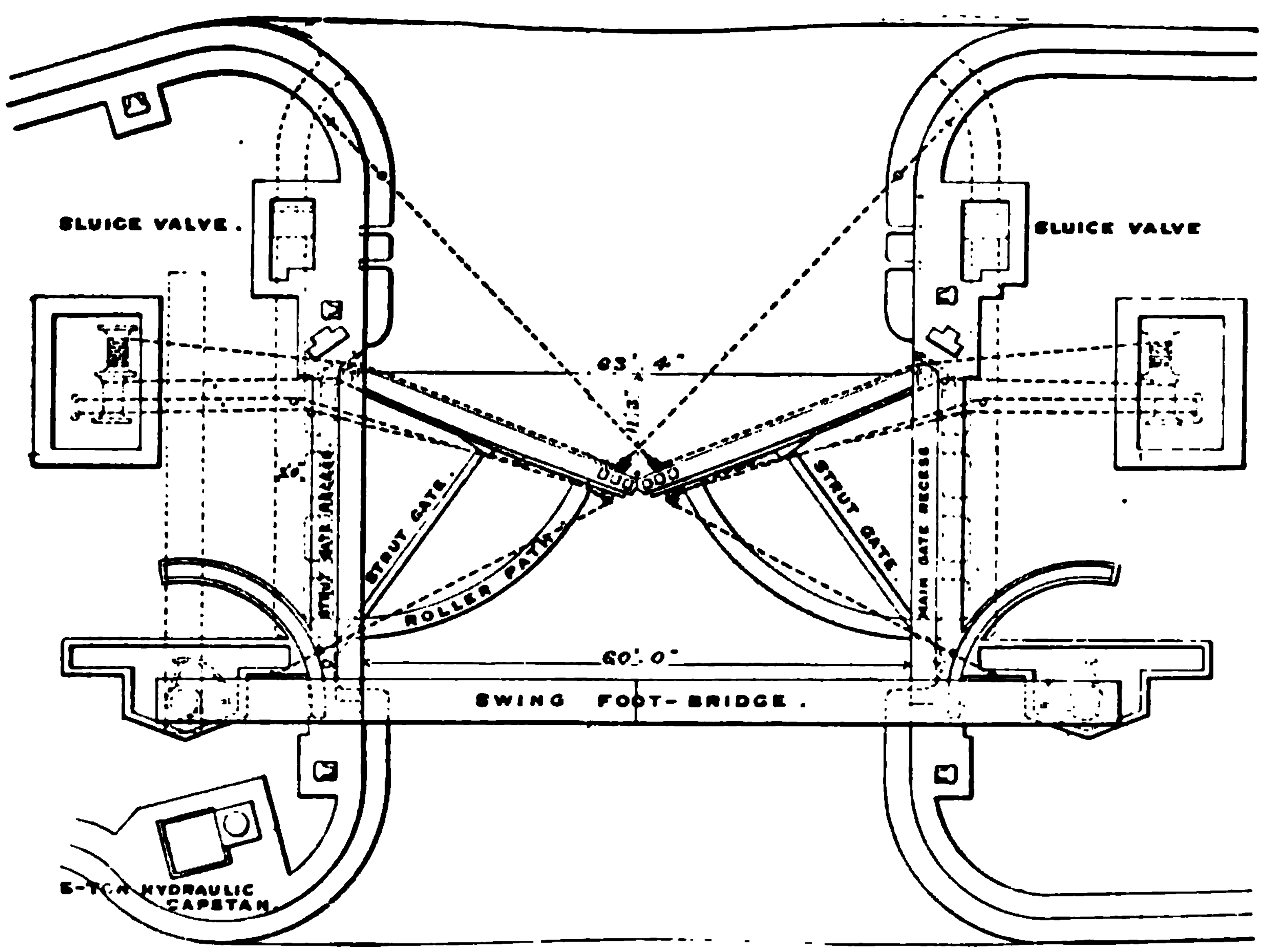

Fig. 201.—Entrance to Eglinton Dock, Ardrossan.

each side of the entrance are 3 feet wide and 4 feet high, with inlet sluices, 2 feet wide and 2 feet high, at the bottom of the gate recess. The sill-stones, hollow quoins, and sluice chamber guides are of granite, the rest are built in rubble concrete, except the sill, gate floor, and aprons, which are of concrete.

* Robertson on "Ardrossan Harbour Extensions," *Min. Proc. Inst. C.E.*, vol. cxx.

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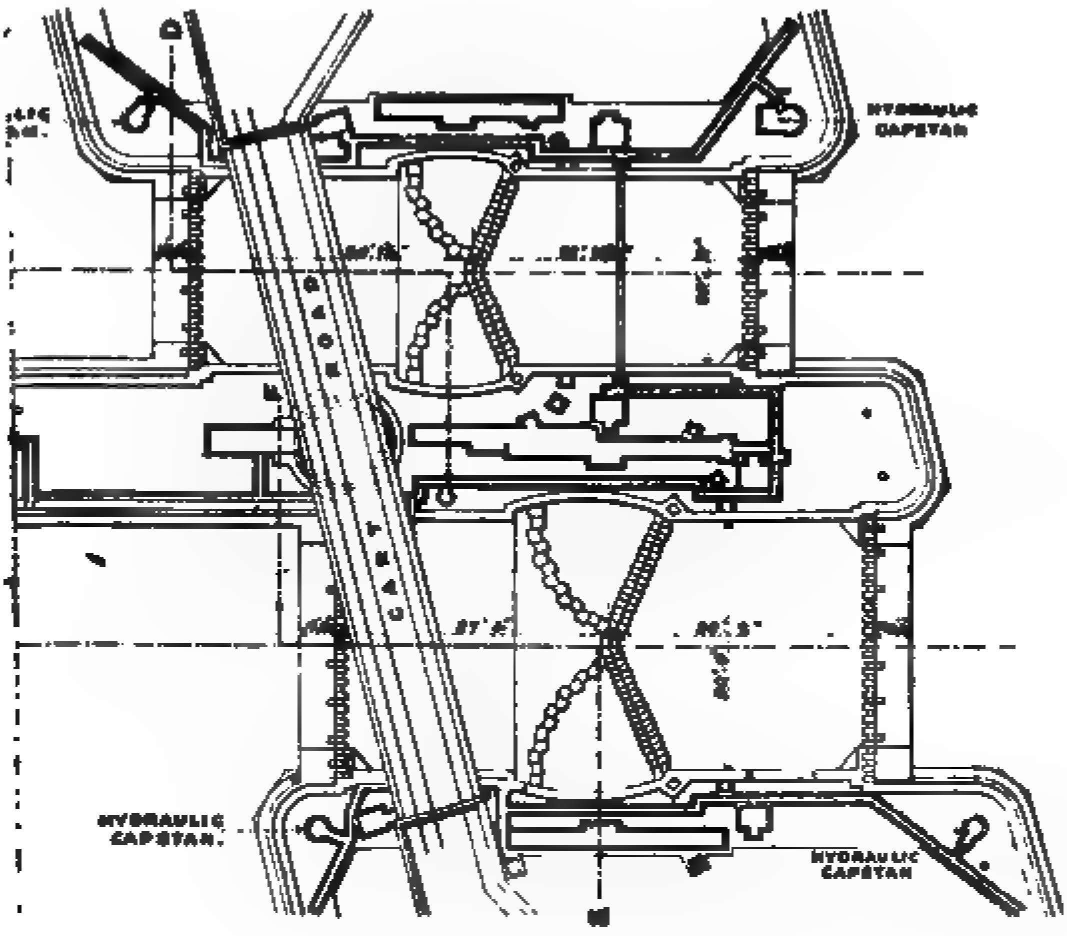


PASSAGE, KIDDERPUR DOCKS

d Locks.

The Alexandra Lock, Hull.*

This lock is 85 feet wide and 550 feet long, divided into two compartments of 325 and 225 lineal feet respectively. The filling and emptying of the lock are done by means of two pairs of 5-foot culverts, constructed in the walls at low-water level. On one side of the sill these culverts open into the gate recesses, and their inlets are closed by external paddles of greenheart, resting against granite faces and worked by hydraulic power. The two outer sills are 18 inches deeper than the inner one. The outer gates weigh 176 tons, exceeding the weight of the inner gates by 6 tons. The roller path is of cast steel, accurately and carefully bedded on granite.

The difficulties encountered during the construction of this lock are very instructive. The site was covered with *warp* or river mud, which was stiff and sticky inshore, but softer further out, encrusting in drying, with a soft interior. It varied in thickness up to 27 feet, and below, beds of warp, sand, gravel, clay, and peat were met with in no definite order.

The foundations of the lock were designed to be laid at 48½ feet below high-water spring tides, or 32½ feet below the level of the mud. The walls at the south end of the lock were commenced in deep trenches, owing to the impossibility of excavating the fluid mud in the open. In the western trench, clay was not met with until 51 feet below high-water spring tides, or 2½ feet deeper than was anticipated. A "blow" occurred at one point, but was promptly remedied before any extensive disturbance could result. An iron pipe was placed in the hole, and surrounded by chalk rubble, filling the hole to the surface level of the clay, which was then covered over with Portland cement concrete in bags, upon which the foundation concrete was laid. A good deal of fine silt was, at first, brought up with the water, but, eventually, the effluent became quite clear, and was led away in a horizontal pipe to a pumping well, the vent being kept open to the last. One or two other cases occurred and were similarly treated. The source of the leaks was practically only the water contained in the strata, the connection with the river, as indicated by the variations in tidal level and head, being very remote.

At the north end of the lock the foundations caused more serious trouble. After clay had been reached through remarkably dry excavation, the bottom of the trench suddenly began to heave, and water burst up in several places in such quantities as to master the pumps. Additional pumping power failed to make any impression. The sides of the trench began to be undermined by the escape of silt; the ground settled, and large holes appeared in the vicinity. These last were staunched with clay puddle, stable litter, straw, and bags loosely filled with Portland cement concrete. Soundings showed a layer of silt, 5¾ feet thick, at the bottom of the trench, while a 40-foot rod failed to reach the bottom of the blowhole. The total collapse of the trench was threatened, so that strong lacings had to be

* Hurtzig on "The Alexandra Lock, Hull," *Min. Proc. Inst. C.E.*, vol. xcii.

inserted and other preventive steps taken. Pumping was reduced to the minimum necessary for getting in a piled foundation for the side walls at the highest possible level. The holes were filled with chalk rubble and the whole area covered with it in order to intercept the flow of silt. Bearing piles were then driven between a network of temporary timbering, connected at the top by whole timber caps and covered with a double thickness of elm planking. As regards the origin of the water in the blows, investigations seemed to indicate the existence of parallel water-courses below the bed of clay running transversely to the lock.

The principal difficulty being anticipated at the inner gate platform, it was proposed to excavate the foundation in small areas, enclosed by half timber sheeting, grooved and tongued, but after a few piles had been driven some blows occurred at the surface, which was a little above dock bottom, and water came up in considerable quantities. Large holes formed, and some of the sheeting disappeared. Cast-iron pipes were driven vertically into the two principal springs, and in one of these the water reached a height of 14 feet above dock bottom. Several hundred yards of clay puddle were absorbed by one hole alone. To reach the origin of the disturbance it was clearly necessary to carry the sheeting lower down, and accordingly pitch pine piles, 14 inches square and 50 feet long, grooved and tongued, were driven so as to enclose the disturbed area and cut off the flow of water, which was effectively done and the foundations completed. The roller path stones and sills were laid on elm platforms over bearing piles. The discharge through one of the blowhole pipes was stopped, but the water continued to flow through the other until the pipe was closed at the completion of the works.

New Lock at Bremerhaven.

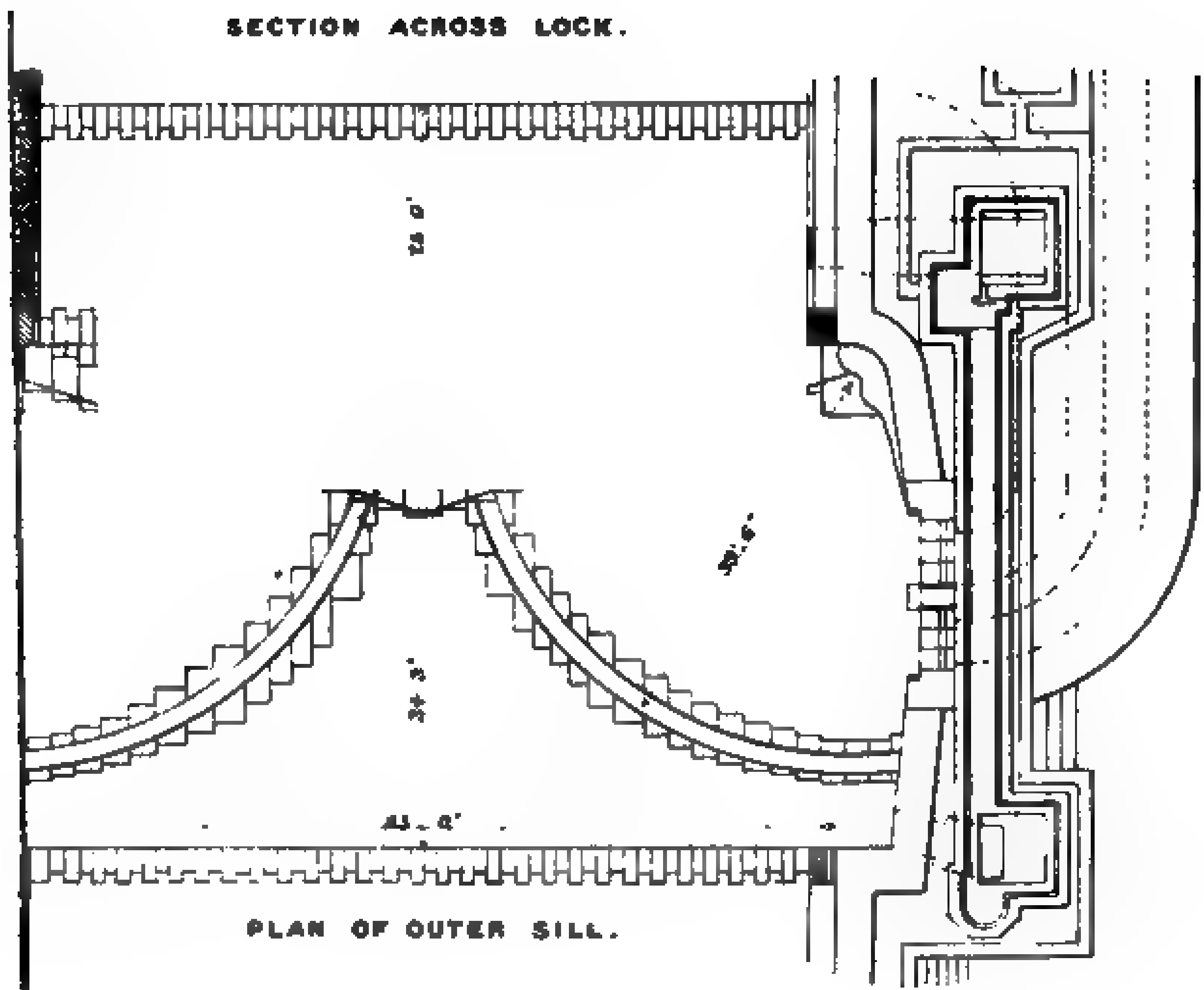
This lock (fig. 206) has an efficient length of 656 feet, or a length of 705 feet between gates. The breadth of the entrance is 92 feet, and of the

CROSS SECTION.

Fig. 206.—Bremerhaven Lock.

chamber 147 feet, so that the largest passenger steamers can lie there preparatory to starting and receive cargo from lighters. The depth is sufficient to accommodate ships drawing 31 feet during neap tides. An

[To face page 266.



dra Lock, Hull.

invert in the floor of the lock has been dispensed with as unnecessary, since no springs were likely to be found in the stiff clay on which the lock stands. The walls, which contain the levelling culverts, are founded on inclined piles, in rows, 4 feet apart. They are inclined alternately in opposite directions, an arrangement which secures a favourable distribution of the forces acting on the piles, and has the further advantage that the pile-heads are not so near together, and the piles can consequently be driven deeper into the solid ground. The inner end of the lock is closed by a sliding caisson, the outer end by a pair of iron gates. The former was selected on grounds of economy and utility as a movable bridge, the latter by reason of their greater strength, for during spring tides a strong current flows through the lock into the Kaiser Dock, which during southerly winds is considerably increased by the heaping up of the tide on the Bremerhaven shore. This current, aided by the force of the waves and the pressure of the wind, exerts a force which, it was considered, could not be so well resisted by a sliding caisson, supported at one end only, as by two strong gates.

CHAPTER VII.

JETTIES, WHARFS, AND PIERS.

DEFINITIONS—STRESSES—WAVE ACTION—FORCE OF IMPACT—RESULTS OF IMPACT—OBSERVED PRESSURES—INSTANCES OF WAVE ACTION—DESIGN OF JETTIES AND PIERS—CONSTRUCTION—CONCRETE MASS, BAG, AND BLOCK WORK—DRESSED MASONRY AND RUBBLE MOUNDS—FASCINE WORK—OPEN TIMBER FRAMING AND CRIB WORK—COLUMNAR STRUCTURES AND FRAMEWORKS OF IRON AND STEEL—MONIER AND HENNEBIQUE SYSTEMS—TYPICAL EXAMPLES AT ABERDEEN, ZEEBRUGGE, HAVRE, KINGSTOWN, ALGIERS, HOOK OF HOLLAND, BLYTH, LIVERPOOL, NEWCASTLE, SOUKHOUM, TOUAPSÉ, BELFAST, DUNDEE, DUNKIRK, TILBURY, MADRAS, SUNDERLAND, GREENOCK, AND HULL.

IN one sense, and that perhaps the most important, jetties, wharfs, and piers may be looked upon as constituting the outlying or advance works of a dock system. It is quite true that they are by no means exclusive, or even indispensable features, being found at many ports which have no docks and absent from others where docks are numerous. Furthermore, they do not always, or necessarily, occupy outlying positions, being often located in sheltered basins and even within docks themselves.

Seeing, however, that their most important functions are discharged in connection with exposed situations, we shall deal with them mainly from this standpoint, and afterwards consider their adaptation to more sheltered areas. And as to the strict propriety, or otherwise, of treating such structures as forming an integral part of a dock system, we need not concern ourselves too closely. The fact that they do play so prominent a *rôle* in many cases, and that they have indubitably demonstrated their ability as accessory features generally, is sufficient justification for treating the subject in its broadest aspect.

Definitions.—Our first duty is a delimitation of the respective constituents of the group.

It is no easy matter to draw a strict, or even a serviceable, distinction between the various types. A *jetty* is radically that which juts out or projects, and the term is appropriately applied to all structures which project from the general contour of any littoral. But it shares this signification in common with piers and moles, both of which are similar projections. The primary meaning of the word *pier* is apparently connected with the notion of support, and it is commonly used in engineering to indicate the intermediate props or supports of a series of arches. Probably from this association, an idea of isolation or detachment has been

acquired, and hence its application to maritime structures, the connection of which with the mainland is of a slight and restricted nature. This feature, however, is equally characteristic of jetties and moles. The word *mole* is evidently derived from the Latin *moles*, a mass, and is indicative of a large mound, or long ridge of material, heaped more or less regularly, in such a way as to constitute some protection from rough external seas. In this respect it fulfils the functions of a breakwater, with which it is closely allied, though, in later times, it has acquired the special significance of a breakwater provided with a broad superstructure capable of being used as an ordinary quay. Perhaps the position may be best summarised thus:— Outlying works in exposed situations, used for protective purposes alone, are breakwaters. When joined to the shore, and equipped for commercial operations, they become, almost indifferently, piers, jetties, and moles. Accordingly, the latter terms will be employed, in the present chapter, as practically synonymous.

A wharf may be defined as a continuous structure, occasionally acting as a retaining wall, along the open margin of a sea, or along the banks of a river, canal, or other waterway. The application of the word is somewhat loose, and it is sometimes taken as identical with quay, though its use in connection with dock and basin walls is rare. Wharfs have obviously provided the most natural sites for the berthing of vessels from the earliest times, being employed for this purpose long before the ideas of outlying jetties and enclosed basins were conceived. In this connection, they are subdivisible into two classes—legal wharfs and sufferance wharfs. The former are certain wharfs, in all seaports, at which goods were required to be landed and shipped by Act 1 Eliz., cap. 11 (now repealed), and subsequent acts. Some wharfs, as at Chepstow, Gloucester, &c., are deemed legal from immemorial usage; others have been made legal by special Acts of Parliament. Sufferance wharfs are places where certain goods may be landed and shipped—as hemp, flax, coal, and other goods—by special sufferance, granted by the Crown for that purpose.* These legal distinctions, however, have no bearing on the engineering aspect of the question.

From their close relationship to ordinary quays, much that has been said in Chapter v. is equally applicable to wharfs, but need not be repeated here.

As part of a dock system, external jetties and piers serve a twofold purpose. In the first place, they act as protective works, by means of which vessels are guided and sheltered during their entry. Secondly, they serve as directive agencies for the deflection or regulation of currents. Whether intentionally on the part of the designer or not, this second function is one which must inevitably be performed by any artificial projection beyond the normal contour of a littoral. Hence it behoves the engineer to exercise great care in determining the location and disposition of a proposed jetty or

* Dr. Ogilvie.

pier, lest serious or even disastrous consequences ensue. The effect of a misplacement might be the shoaling of a hitherto navigable channel.

Unfortunately, the conditions affecting fluvial, estuarine, and marine currents are too complex for anything of the nature of a brief and satisfactory *resumé*, and the subject, indeed, constitutes a branch of maritime engineering which scarcely comes within the purview of the present treatise. There can, however, be no doubt that the influence of training works, in the form of walls and dykes, upon the augmentation and maintenance of waterways is very powerful, and that, judiciously employed, they are a valuable means for increasing the accessibility of a port. There are several instances of such works in existence, notably at the mouths of the Tees and the Ribble, where training works have been recently constructed in order to afford a navigable channel, in the first instance to the ports of Middlesbrough and Stockton, and in the second to the town of Preston.

External jetties are either detached or arranged in pairs. Of double jetties there are three forms—viz., parallel, convergent, and divergent. Parallel jetties are mainly used for training purposes, as at Leith and Ostend; convergent jetties enclose a sheltered basin or outer harbour, as at Barry and Sunderland; divergent jetties afford guidance and direction to vessels entering a narrow waterway, as at the Alexandra Dock, Hull, the Canada Basin, Liverpool, and the Tilbury Docks, London.

Jetties are also to be found in the interior of many docks, especially those of large size, with the object of increasing the proportion of quayage to water area. Thus the south-west India Dock, London, with an area of 26¾ acres, is furnished with 16 jetties, affording accommodation to 32 vessels. The Victoria Dock, at the same port, has 13 jetties in an area of 74 acres. The Alexandra Dock, at Hull, has 4 jetties in an area of 46½ acres.

Wide projections, of solid construction, into the interior of a dock are designated *Tongues*, such as the Canada Tongue at Liverpool. They really constitute an integral part of the dock outline.

Short tongues or jetties used for coaling purposes are called *Staiths*. There are 13 staiths at the Penarth Dock and 31 at Barry Docks.

Stresses in Piers and Jetties.—In Chapter vi. some consideration has already been given to the character and influence of various natural agencies in so far as they affect the navigability and usefulness of dock entrances. In the present section it will be necessary to supplement this information by some observations on the effect of these agencies upon the stability and durability of exposed structures.

Wave action alone calls for serious notice. The effect of wind pressure upon the superstructure of a pier is trifling compared with that of the onset of waves upon its base. The only danger to be apprehended from currents is their tendency to undermine the foundations, and this can readily be guarded against by the exercise of the precautions indicated in Chapter v.

The mathematical theory of waves is a physical question too purely academic for discussion in an engineering treatise. Students who desire

information on the subject are referred to the articles on "Wave" in each of the Encyclopædias, Britannica and Metropolitana.

The points which more immediately concern the engineer are the nature, direction, and magnitude of the disruptive forces, as determined by actual observation.

Although waves have been divided into two classes—those of oscillation and those of translation—it is probable that all waves are more or less waves of translation, causing the particles of which they are composed to move forward horizontally to some extent. Certainly this is the case with all large and important waves affecting the stability of maritime works.

When a wave advances into water which becomes increasingly shallow, its energy is communicated to successively decreasing masses, and there is consequently a tendency to produce in those masses a greater and more violent agitation; but this effect is generally diminished, and sometimes entirely counteracted, by the loss of energy due to friction along the bottom, and to eddies and surging.

The bottom friction produces a distortion of the elliptical orbits of the particles of water, causing the crest to advance more quickly than the trough. At length the crest overhangs the face slope, falls forward, and breaks into surf. At this point the forward motion of the particles is equal to the velocity of the wave, and the stroke represents the maximum effort of the latter. Now, the velocity of a wave in shallow water is found to be nearly the same as that which would be acquired by a heavy body in falling freely from rest, under the action of gravity, through a height equal to the semi-depth of the water plus three-fourths of the height of the wave. Accordingly, we have

$$v = \sqrt{g\left(d + \frac{3h}{2}\right)}, \quad . \quad . \quad . \quad . \quad (44)$$

where v is the velocity of the wave, h its height, and d the depth of the water.

When the depth of water exceeds the length of the wave, the speed of the latter is practically independent of the depth, and is almost exactly equal to the velocity acquired by a body falling through half the radius of a circle whose circumference is the length of the wave.

The reaction of a surface subjected to the force of continuous impact is measured by the rate at which momentum is destroyed. Hence, if w be the weight of a unit volume of water, wv is the mass which impinges on unit surface in unit time, and wv^2 is therefore the amount of momentum. And since the weight of 1 lb., falling freely, generates in one second g units of momentum, the reaction of the surface will be equivalent to a weight of $\frac{wv^2}{g}$, and this represents the pressure per unit area due to the impact.

Lieut. Gaillard (Corps of Engineers, U.S. Army), has demonstrated by experiments upon small areas that the maximum intensity of force in breaking waves in such cases occurs at a level slightly above still water,

diminishing to zero at the crest, and to one-half the maximum at the bottom. But like wind-pressure data, results on small areas are no guide to stresses over extensive surfaces.

If a wave, before breaking, reaches a wall or other obstruction having an abrupt, vertical face, it is reflected in the following manner:—The particles of water in contact with the wall move up and down through a height double the height of the original wave. At a distance away from the wall equal to a quarter of the length of the wave, the particles move horizontally backwards and forwards. Between these two points the motion of the particles is a compound one, and movement takes place at various angles.

Consequently, the action of waves upon a pier or jetty must be taken as resulting in the creation of four distinct forces:—

1. A direct horizontal force exerting compression.
2. A deflected vertical force, acting upwards and tending to shear any projections beyond the surface of contact.
3. A vertical downward force upon the base of the wall, due to the collapse of the wave.
4. The suction of the back draught upon the foundation.

Apart from the hydrostatic pressure, augmented to a very considerable degree by the force of impact, the following subsidiary results will take place, viz.:—

1. A vibration of the structure tending to weaken the connection of the various parts.
2. A series of impulses imparted to particles of water contained in the pores and joints of the structure, producing internal pressure in various directions.
3. The condensation and expansion of air confined in cavities and interstices, causing disruption.

It is impossible on any purely theoretical basis to determine with the least degree of accuracy and precision the magnitude of these various stresses. Practical observation must therefore be called in to supply the deficiency, by providing data as to the maximum stresses likely to be encountered. Investigations have been made in several instances with the aid of a marine dynamometer, devised by Stevenson, with the result that in the most exposed cases, the pressure registered did not exceed 3½ tons per square foot. With waves 10 feet high, a mean pressure of 1·36 tons was indicated. Other instances are as follows:—

At Skerryvore, from 2½ to 2¾ tons per square foot.
At Bell Rock (German Ocean), 1½ tons per square foot.
At Dunbar (East Lothian), 3½ ,, ,, ,,
At Buckie (Banffshire), 3 ,, ,, ,,

Experiments by Mr. Frank Latham, at Penzance, showed a pressure of 18 to 20 cwts. per square foot, at right angles to a sea wall, in 10 feet of water, with a wind pressure of 15 to 18 lbs. per square foot.

At Cherbourg, the force of waves in storms has been found to vary from 600 to 800 lbs. per square foot.

Instances of Wave Action.—The following are a few recorded instances of the feats performed by waves :—

During a summer gale, in the year 1869, fourteen stones, each 2 tons in weight, part of the structure of the Dhu Heartach Lighthouse, which had been laid in Portland cement and fixed in their places by joggles, at a level of 35 feet 6 inches above high water, were torn up, and eleven of them swept off the rock into deep water.*

During the storms of December, 1896, and January, 1897, blocks, weighing 40 tons each, used in the construction of Peterhead breakwater, were displaced in courses bedded respectively at the levels of 17 feet $1\frac{1}{2}$ inches and 23 feet $7\frac{1}{2}$ inches below low water of spring tides. One of these blocks lodged on a concrete platform, 30 feet 7 inches below low water, and was washed away during a storm in the following March.†

The destruction of the outer extremity of the breakwater at Wick, in December of the year 1872, is described in a report by Messrs. Stevenson to the directors of the British Fishery Society.‡ The end of the work was protected by a mass of cement rubble work. It was composed of three courses of large blocks of 80 to 100 tons, which were deposited as a foundation on the rubble. Above this foundation there were three courses of large stones carefully set in cement, and the whole was surmounted by a large monolith of cement rubble, measuring about 26 feet by 45 feet by 11 feet in thickness, and, at 16 feet to the ton, weighing upwards of 800 tons. This block was built *in situ*. As a further precaution, iron rods, $3\frac{1}{2}$ inches in diameter, were fixed in the uppermost of the foundation courses of cement rubble. These rods were carried through the courses of stonework by holes cut in the stone, and were finally embedded in the monolithic mass which formed the upper portion of the pier. Incredible as it might seem, this huge mass, weighing not less than 1,350 tons and presenting an area of 496 square feet to the sea, was gradually slewed round by successive strokes until it was finally removed and deposited on the rubble inside the pier, having sustained no damage beyond a slight fracture at the edges. The lower or foundation course of 80-ton blocks, which were laid on the rubble, at a depth of 15 feet below low water, retained their positions unmoved. The second course of cement blocks, on which the 1,350 tons rested, was swept off after being relieved of the superincumbent weight, and some of the blocks were found entire near the end of the breakwater.

The displaced mass was succeeded by a still more enormous block, weighing no less than 2,600 tons, which, after remaining undisturbed for

* Stevenson on "The Dhu Heartach Lighthouse," *Min. Proc. Inst. C.E.*, vol. xlvi.

† Shield on "The Effects of Waves on Breakwaters," *Min. Proc. Inst. C.E.*, vol. cxxxviii.

‡ Vide *Min. Proc. Inst. C.E.*, vol. xliii.

three years, was carried away bodily by a storm in January, 1877, and deposited in two pieces within the line of the breakwater.

But even this is not the limit of wave power. During the storm of October, 1898, which is said to have been as severe as any that have been witnessed in Peterhead Bay, the waves were 30 feet in height, and a section of the breakwater there, down as far as 10 feet $7\frac{1}{2}$ inches below low water and weighing 3,300 tons, was bodily slewed to the extent of 2 inches, without the brickwork being dislocated. This enormous mass slid upon the surface of the course immediately below it, the blocks in which were, strange to say, quite unmoved. In the waves which were responsible for this feat, the water was thrown up to a height of about 115 to 120 feet, and the surface upon which they acted measured 33 feet by 44 feet, or 1,122 square feet. In order to form an idea of the force required to slew such a mass, the Engineer, Mr. William Shield, ascertained the coefficient of friction of blocks similar to those forming the breakwater, by causing them to slide upon a concrete floor. The floor was well wetted, and the average of several trials with blocks up to 68 tons weight, gave a coefficient of 0·7. In moving the mass, the waves must therefore have exerted a force of 2,310 tons over the whole area exposed to them, or slightly over 2 tons per square foot. Although about one-third of the mass was below the level of low water, the troughs of the waves would be considerably below its lowest point, and taking all the circumstances into consideration, little, if any, allowance need be made for flotation. If such allowance, however, be considered necessary, it is probable that some deduction should also be made from the area exposed to the wave-stroke, so that the above force per square foot would not be much affected.*

After this incident it is, perhaps, not surprising to find that a 20-ton block at Ymuiden breakwater, in Holland, was lifted to a height of 12 feet vertically up the face of the pier and landed on the top of it.†

The Design of Jetties, Wharfs, and Piers.—The principles of the stability of quays have already been set forth, and they are equally applicable to those wharfs of solid construction which act as retaining walls. The depth of a wharf or river wall, however, will generally require to be greater than that of a dock wall, on account of the vertical disturbance of vessels by waves. Open timber wharfs in front of pitched slopes, allow the waves to pass through and expend themselves upon the bank, so that the wharf structure does not encounter the full force of the waves, but this arrangement is only feasible in situations where the exposure is not great.

In considering the stability of structures subjected to external forces of great magnitude, it will be found that there are two distinct sources of resistance, upon either of which a design may be based—viz., the resistance

* Shield on "The Effect of Waves and Breakwaters," *Min. Proc. Inst. C.E.*, vol. cxxxviii.

† *Ibid.*

due to the inertia of a solid mass and the resistance offered by the inherent strength of a scientifically framed structure. The first case is exemplified by piers constructed in huge blocks of masonry and concrete, and in the second by trussed open work piers of timber, iron, or steel. Nature, it is to be noted, opposes the violent onset of stormy seas with huge boulders and rocky headlands, and accordingly such natural features constitute an obvious type of massive construction. Framed structures, on the other hand, represent the result of human thought and adaptation. Theoretically, both principles would seem to be equally effective, but in practice it will be realised that the joints in framed structures are a source of weakness, owing to their tendency to loosen under vibration; and further, that there is the very important factor of deterioration and decay, which gives a decided advantage to the employment of a practically indestructible material, such as stone or concrete, over less durable substances, such as timber, iron, and steel. In the latter cases, there must be a constant expenditure on maintenance and repair.

Where there is an important littoral current, which it is undesirable to divert in any way, the use of columnar piers becomes a necessity. The current then passes through the openings without perceptible obstruction.

Construction of Jetties.—Jetties, wharfs, and piers, considered as forming a single class, may be constructed on any of the following systems, either singly or in combination :—

Concrete,	Mass work. Bag work. Block work.	*Timber,*	Fascine work. Open framework. Crib work.
Stone,	Dressed masonry. Rubble mound.	*Iron and Steel,*	Columnar structures. Close framework.
Composite,	Monier system. Hennebique system.		

It will be useful to deal with the salient features of each of these various systems *seriatim*.

Concrete Mass Work consists in the deposition of a large bulk of fluid concrete within an enclosure, formed either by a boundary of sheet piling or by temporary retaining moulds, which latter are removed when the concrete is sufficiently set. The method is not, generally speaking, satisfactorily adapted to subaqueous construction, as, apart from the awkwardness of setting wooden moulds under water, it is difficult to prevent excessive dilution and washing away of the cement particles, whereby the strength of the concrete is seriously impaired. Accordingly, the method is mainly restricted to situations in which it can be carried out in the open—that is to say, either above low-water line or, when below that level, by tidework and within the shelter afforded by cofferdams. Notwithstanding this, there are undoubtedly instances in which fluid concrete has been successfully deposited under water, but the local conditions in such cases have been peculiarly favourable. One of the main elements of success is perfectly quiescent water. Where the water level fluctuates rapidly and erratically, as in an

exposed tidal way, with its attendant ground-swells and rapid currents, the risk is sufficiently great to render other methods preferable.

Concrete Bag Work, introduced in 1865 by Mr. P. J. Messent for the purpose of repairs at Tynemouth, and developed into a system of subaqueous construction about the year 1870 by Messrs. Cay and Barton at Aberdeen and Greenore respectively, consists in filling jute bags with fluid concrete and depositing them immediately *in situ* with the aid of divers. If the work be carried out expeditiously, before the concrete has had time to set, the bags will adapt themselves to the inequalities of the surface upon which they are laid, and so ensure a complete and uniform bearing for each successive course. The size of the bags used in various instances, ranges from a capacity for 5, to one for 100 tons of concrete, or even more. The material used is jute sacking, weighing from 25 to 30 ounces per superficial yard. The bags, after being filled at the mixing station, are conveyed to their respective positions and lowered in wrought-iron skips, through the hinged bottom of which they are discharged. Adjustment and flattening is performed by the divers. As there is a tendency for the exposed ends of the outermost bags to break away under heavy wave action, it is advisable to construct the work slightly wider than the nett width desired. Bag work forms an admirable method of dealing with irregular foundations too indurated for dredging, such as hard rock and clay containing massive boulders.

Concrete Block Work is an adaptation of the principles of masonry on a large scale to concrete construction. The blocks are prepared on shore in the ordinary way, by means of wooden moulds of the shape required. For foundation and interior work the rectangular or square form is the most usual. The blocks are of any convenient size, ranging from 5 tons to a weight limited only by the power available for lifting and depositing. In order to facilitate setting, each block is sometimes constructed with two vertical or slightly inclined perforations, through which are passed iron bars with T or angle ends, capable of engaging against the under side of the block when turned through a right angle. These are, of course, removed after the block has been set. Other appliances for lifting and depositing are illustrated on p. 114, *ante*. Setting operations may be carried out by a floating crane, by a traveller running upon a temporary staging, or by a crane traversing the portion of the work previously constructed and able to set a block some distance in front of its leading wheels. Except in the case of very smooth water, the traveller and the land crane constitute by far the steadier agents. The blocks are set on the outer faces of the structure, and are ranged as closely as possible in order to admit of being connected by cramps and joggles. Where the circumstances render such a process feasible, the joints may be pointed in cement, or, if too wide for this, the openings may be made good with brickwork in cement. The interior of the work will then be filled with blocks, arranged so as to break joint, and well bedded in concrete grouting, which may be run through a pipe under a considerable head after the blocks are set.

All three of the foregoing systems may be, and have been, used in combination, such as, for instance, a construction of block work below low-water level, resting upon a bag-work foundation course, and having a superstructure of mass concrete.

The south breakwater at Aberdeen was carried out in this manner, and as the statement of expenditure affords a comparison of the cost of the several methods, it is appended here:—*

	Cubic Yards.	Expenditure.	Cost per Cubic Yard.
Bag work in foundations, ...	3,202	£4,045	25/3
Block work, including blocks inserted in fluid concrete, ...	22,851	18,175	15/11
Mass concrete in frames, ...	23,356	18,868	16/2

A better appreciation of the relative cost will be gained by a brief statement of the precise conditions obtaining in each case.†

Bag Work.—The bags were deposited by iron skips, the greater part by two skips each holding 5¼ tons of concrete, their inside dimensions being 6 feet by 4 feet by 3½ feet deep. In the last year, a skip of 16 tons capacity was used, its dimensions being 9 feet by 6 feet by 6 feet. The bottoms of the skips opened on hinges, the hook which held them being released by a trigger. In the larger skip the closing of the doors, after the bag was deposited, was assisted by counterbalance weights. The bag, of the same shape as the skip but rather larger, was fitted into it and temporarily lashed at the top so as to line the skip. It was then filled with liquid concrete (1 cement, 2½ sand, 3½ gravel), the temporary lashings removed, and the mouth of the bag sewn up. The skip, with its contents, was lowered by a crane to the divers, and moved about, in obedience to their signals, until close over the required position, when the trigger was pulled by a rope from above, and the bag discharged.

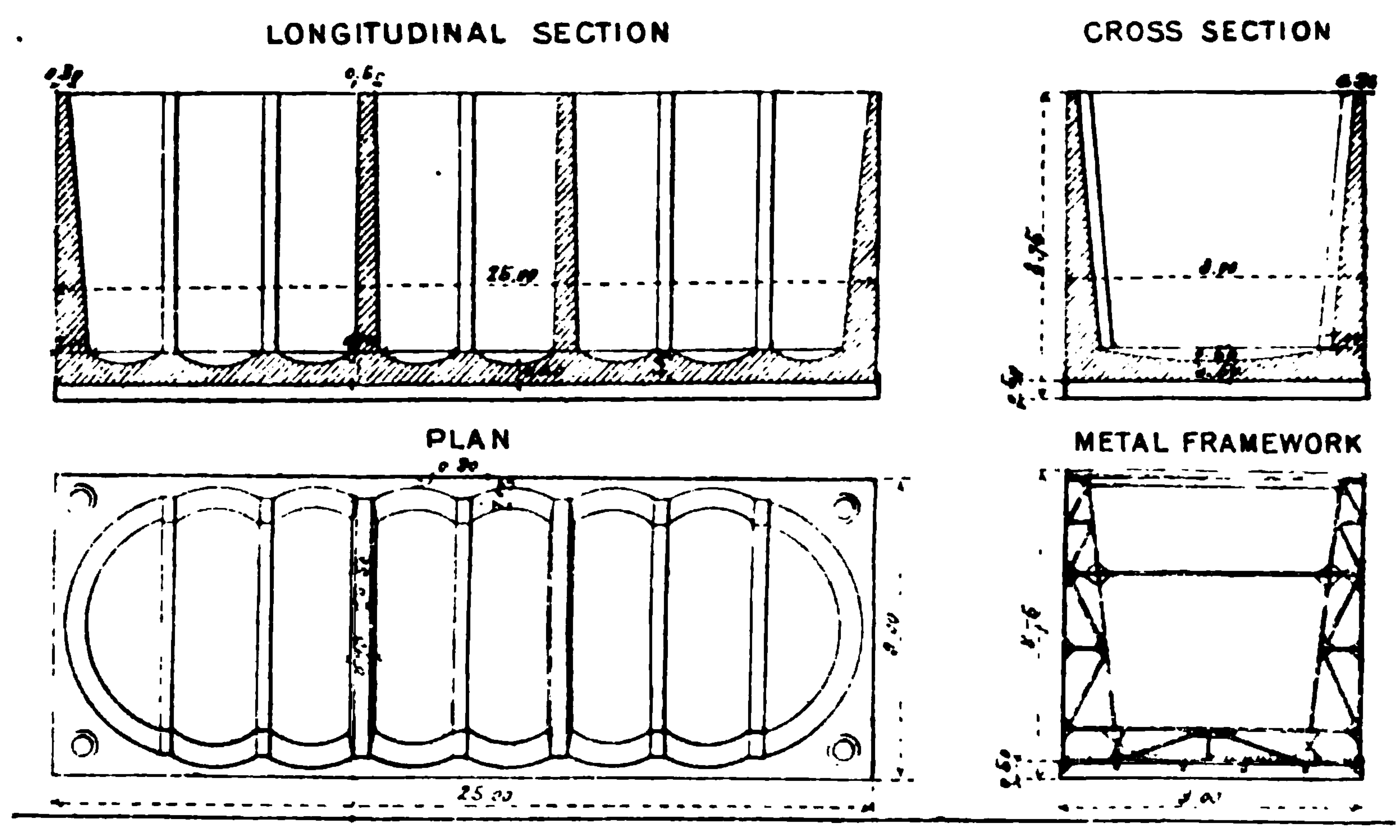
Block Work.—The blocks were all 4 feet high and usually 6 feet wide. At first, they were of sizes varying in weight from 7½ to 18 tons; latterly, the small blocks were mostly used for incorporation among the fluid concrete or mass work, and the larger, from 10½ to 24 tons weight, for block building. The blocks were cast in wooden moulds in the usual manner, the proportions of the concrete being 1 cement to 4 of sand and 5 of gravel, with large pieces of broken stone imbedded. They were staked by cranes in the block-yard to harden, and then taken down an incline, on waggons, to the staging cranes, by means of which they were lowered to and set by the divers.

Mass Work.—A framework of posts was erected round the site of the building, excepting at the ends of the completed work, which formed one

* This statement does not include items for preparatory works, plant, staging, &c.

† Cay on "The South Breakwater, Aberdeen," *Min. Proc. Inst. C.E.*, vol. xxxix.

side of the case. The posts were provided with grooves, into which panels were slid, extending from post to post. The bottom and sides of the case were lined with jute bagging, and tie-rods, passing through the posts and from side to side, prevented the case from being burst open by the lateral pressure of the fluid concrete. The heart of each post was a piece of Baltic fir, 20 feet long by 12 inches by 6 inches, scantling; the pieces of wood for forming the grooves were fixed to the larger sides. The panels were built up of short pieces of plank 2 feet long, placed vertically, so as to form a slab 7 feet 9 inches by 2 feet by 3 inches, and they were backed by two horizontal planks 7 feet 4 inches by 11 inches by 3 inches. The ends of these formed the tongues which slid in the grooves in the sides of the posts. The tie-rods were of wrought iron $\frac{3}{4}$ inch diameter, in convenient lengths, connected by $\frac{3}{4}$-inch shackles. The jute bagging was 39 inches wide and weighed $29\frac{1}{2}$ ozs.; it cost 8d. per lineal yard and could generally be used twice. The proportions of the concrete found best for the work were 1 cement to 3 sand and 4 gravel; much of it was executed, however, in the proportion of 1 cement to 4 sand and 5 gravel.



Figs. 207, 208, 209, and 210.—Caisson at Zeebrugge.

A special adaptation of the concrete block system, as practised in the construction of the outermost portion of a mole at Zeebrugge, merits some notice. It consisted in the formation of hollow blocks of concrete of height sufficient to reach above low water from the ground level. These were floated out into position, sunk, and filled with concrete. The circumstances at Zeebrugge were favourable to this method, the depth of water not exceeding 30 feet at low water and being generally 26 feet.

The blocks, or caissons (figs. 207 to 210), were moulded about an iron frame with plated sides, and were 80 feet long by 30 feet wide by 30 feet deep. This gives a volume of 72,000 cubic feet each, and a total weight of about

4,500 tons. The underside of each caisson had a knife edge to penetrate the ground. The concrete was composed of 33 parts of small stone and 13 of sand to 5 of cement. The caisson was designed with three compartments, and in each of the walls there was provided an orifice for filling them with water. The orifices were temporarily plugged while the caisson was being towed into position. On removing the plugs, the block foundered. The interior was then filled with concrete by means of skips opening at the bottom. The top layer of 3 feet was deposited in the dry at low water, with concrete very rich in cement. Large pieces of rock were then sunk to the seaward of the block, and along its base, to prevent any danger of undermining by the water. The ground was a clayey sand.

Upon the foundation course thus laid, the upper blocks, of 55 tons weight each, were set by a Titan crane. The jetty was constructed with horizontal offsets, in order to partially destroy the downward effect of a breaking wave upon the foot of the wall (fig. 211).

SECTION OF THE BREAKWATER JETTY

Scales.

0 5 10 Metres

0 10 20 30 Feet

Fig. 211.—Jetty at Zeebrugge.

Other examples of bag work are to be found at Sunderland (figs. 244 and 245), of block work at Dover (fig. 213), and of mass work at Liverpool (figs. 221, 222, and 223). The subject of concrete work has also been treated in the chapter on Dock Walls, and instances are there given of quays constructed on the same or kindred lines.

Masonry Piers are not so common as they used to be in the days before the introduction of cement concrete. They are only executed now in places where suitable stone is very plentiful and skilled labour cheap. In other situations, concrete offers every inducement for its adoption. Masonry piers usually have facings of ashlar with heartings of rubble,

though in some cases, pockets of earthwork have been employed. With either system of construction, it is essential for the stability of the work that the opposite facings should be securely tied together by well bonded cross walls, or by horizontal lacing courses at regular intervals. The largest possible stones should be chosen for the outer blocks, and they should be secured to one another by dowels and plugs as well as dovetailed

Fig. 212.—Pier at Havre.

into the hearting by an efficient system of bonding. The south pier at Havre (fig. 212) is a typical example of masonry construction. It has inclined ashlar facings, averaging 5 feet in thickness, connected, at intervals of 5 feet in height, by lacing courses, 2 feet thick. The bottom width is 36 feet 6 inches, and the top width, between parapet walls, 18 feet 6 inches. The pavement is 7 feet 6 inches above high water of equinoctial tides, and 33 feet 4 inches above ground level.

A combination of a granite ashlar facing with a hearting of concrete blocks is exemplified in a pier at Dover, constructed about the year 1855. Present practice at that port favours the concrete block system throughout, with a thin facing of granite rubble above low-water level (fig. 213).

Fig. 213.—Jetty at Dover.

Piers wholly of loose rubble are indistinguishable from breakwaters, their principal function being the destruction of waves. There are but few instances of such works being used for landing purposes. There is one, however, at Kingstown Harbour, near Dublin, where a long inclined mound of loose rubble, with slopes ranging from 1 to 1 to 5 to 1, is crowned with a pitched surface on the inner side, 38 feet in width. The maintenance of such disorganised masses is apt to be costly, as they suffer considerably from the effects of wave action.

Since upright piers from the sea bottom are inevitably expensive in construction, where the depth is at all considerable, and further, since the rubble mound offers a suitable means of bringing the foundation level tolerably near the water level without incurring too much danger of disturbance, a combination of the two types is a very common feature of modern practice.

The level at which loose rubble of different sizes may be trusted to remain stationary in stormy weather is a matter of considerable importance in piers of this type. Sir John Coode states that he found the shingle of Chesil Beach in motion during winter storms, at a depth of 8 fathoms. The line of permanent mud, which marks at any rate the extreme limit of wave action, whatever other agencies may assist in its determination, lies at a depth of 12 to 16 fathoms below low water off the coast of Holland, and at

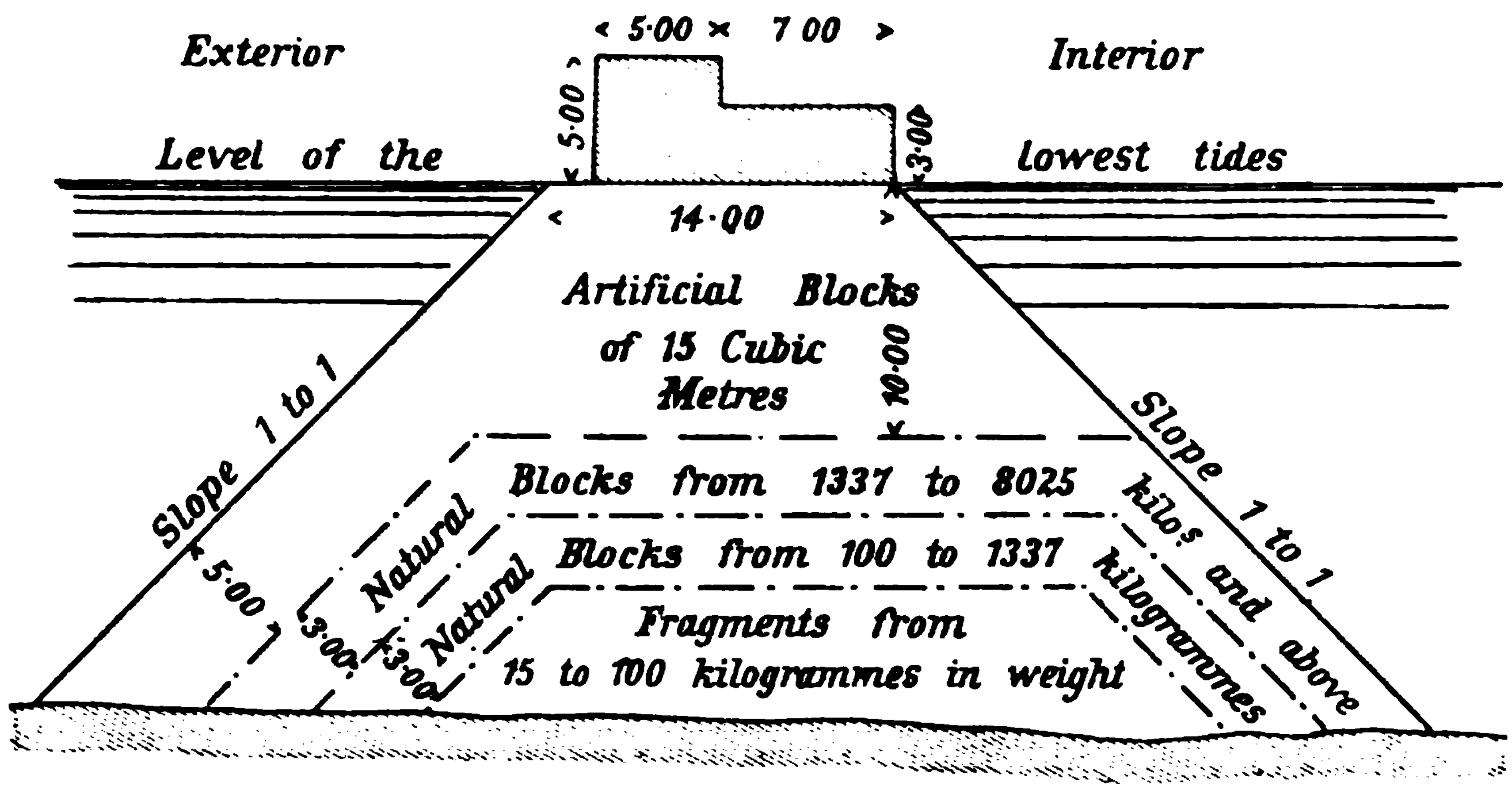


Fig. 214.—Jetty at Algiers.

a depth of 80 to 90 fathoms in the vicinity of the Shetlands. But even assuming the motion of waves to be perceptible throughout so great a range, it is manifest that the force diminishes with the distance below the surface, and that, at a certain depth, the effects become of trifling importance. In fact, it appears that the really injurious effects of wave action are confined to a zone extending from the surface level to a distance of about 25 or 30 feet below. Beyond this point, small rubble and quarry rubbish may be deposited, with comparative impunity, in mounds which will stand at slopes of 1 or $1\frac{1}{2}$ to 1.* Upwards of this, stones of larger bulk and greater weight must be employed, culminating in blocks of not less than

* There are, of course, abnormal cases in which these statements do not accord with experience. For instance, at Peterhead Harbour in October, 1898, blocks weighing upwards of 41 tons each were displaced by the waves at a depth of $36\frac{1}{2}$ feet below low water of ordinary spring tides, but this and one or two other examples at Wick and elsewhere are exceptional.

25 to 30 tons weight at the summit. Owing to the difficulty of quarrying such blocks, concrete monoliths are now generally adopted for the uppermost layer. No benefit is derived from any attempted consolidation of the work by intermixing large and small pieces. On the contrary, the result is likely to be harmful, since the dislocation of the larger blocks will be facilitated in consequence of the small pieces getting under and between them. Blocking the interstices with cement concrete, in bags or otherwise, is a much more satisfactory course.

As an illustration of the combined system of construction, we may take the North Jetty at Algiers (fig. 214). The bottom hearting, 16 feet in height, consists of rubble from 30 to 200 lbs. per piece. Overlying this there are two layers, each 10 feet high, of natural blocks, ranging in the lower layer from $\frac{1}{10}$ to $1\frac{1}{4}$ tons, and in the upper layer from $1\frac{1}{4}$ to 8 tons in weight. The remaining distance of $32\frac{1}{2}$ feet to low-water level is occupied by artificial blocks containing about 550 cubic feet. The superstructure is carried to a height of 16 feet.

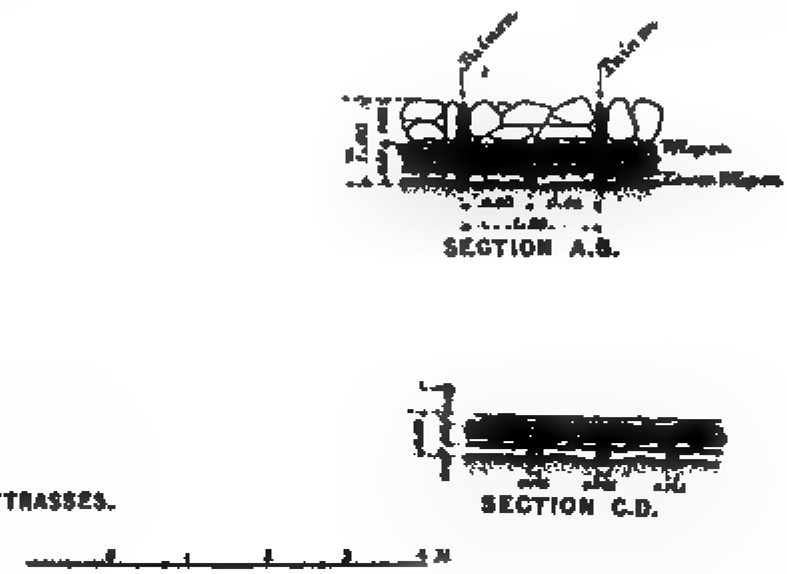
Timber Piers are less substantial than those of masonry or concrete, but they possess certain advantages as regards economy and rapidity of execution. Where the ground is suitable for the reception of piling, and in localities where storms are infrequent and of no great severity, timber jetties and piers can be constructed at a cost much less than that of more massive structures. In ice-bound ports, too, such as those in the Baltic, the prosecution of the work of piling is independent of the season and can be carried on uninterruptedly through the winter, which is an important consideration.

The simplest, and certainly the most primitive, system of timber jetty work is that inaugurated by the Dutch, who build their quays very largely with the aid of fascines (Dutch, *ryshoot*), or bundles of brushwood derived from copses of willows, osiers, &c. Mattrasses of this material, weighted with stone, are sunk in position in successive courses, the whole structure being secured by rows of vertical and inclined piling. The advantages claimed for the use of brushwood are (1) its elasticity, which renders it less liable to injury from the impact of waves, and (2) its solidification under the accumulation of sand and drift in the interstices. To these may be added its convenience and cheapness.

The following particulars relate to the piers at the Hook of Holland, near Rotterdam (see figs. 215, 216, and 217) :—

The piers were constructed of successive layers of *zinkstukken*, or mattrasses, 54·7 yards long by 26·2 yards broad, and 1 foot 8 inches thick, constructed as follows :—Two stakes were driven into the ground, about 2 feet 6 inches apart, to which a cross stick was secured about 2 feet 3 inches from the ground. A series of these frames were erected, 2 feet apart, the number depending on the size of the *zinkstuk*. The fascines were then placed on the cross sticks, being drawn out lengthways, so that each bundle overlapped and bonded well into the next. They were

laid of such thickness that on being bound round in the form of a rope, the circumference was 17 inches. When the full length for one rope, or *wiep*, had been laid out, the fascines were tied at 15-inch intervals with osier bands, tightly twisted and with their ends tucked in. Light intermediate bands, 4 inches apart, were then added. The *wiepen* were next laid in parallel rows upon the ground, about 3 feet apart, to the full width of the proposed mattrass. They were crossed by a second layer at right angles to the first, thus forming a network, which was secured by



Figs. 215, 216, and 217.—Fascine Work.

SECTION OF NORTH MOLE.

Fig. 218.—Mole at Hook of Holland.

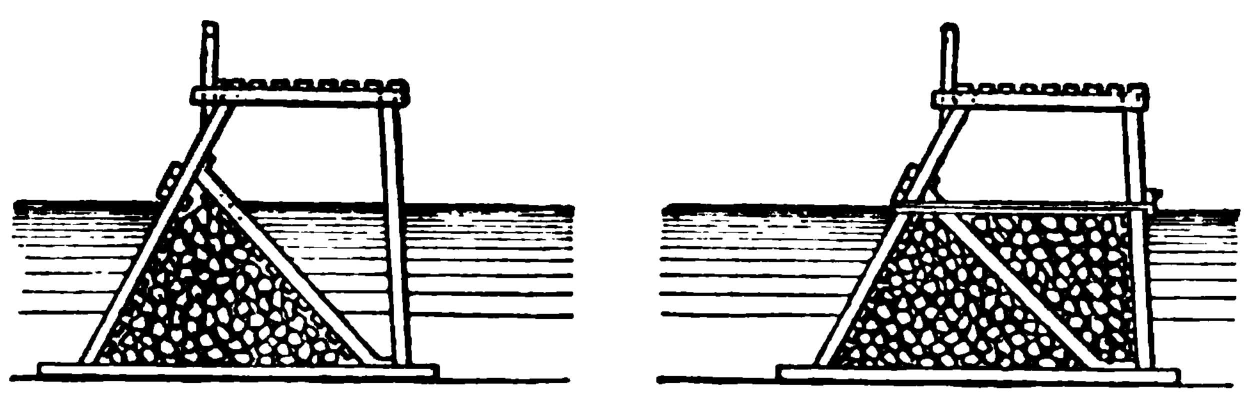
lashings of $\frac{1}{2}$-inch tarred rope with free ends, and withes. Two such networks, upper and lower, enclosed three layers of ryshout, set crossways, 18 inches thick in all, and were tied together by the rope ends. This completed the mattrass. In order to cause sinkage, it was weighted with stone, and the loading was afterwards continued until it amounted to 10 cwts. per square yard. The body of the piers took from five to six mattrasses, averaging with the stones, about 3 feet 3 inches thick; these were further held in place by five rows of piles, driven about 11 or 12 feet

through the mass into the sand below. The outer slopes and edges of the mattrasses were covered with a coating of stone, averaging 13 cubic feet per lineal foot of pier. The part above water was covered with larger stones, retained by rows of small oak piles, the ends of which project above the level of the work, with a view to breaking the force of the waves.

A cross-section of the north pier is given in fig. 218. It has a width of 29 feet 6 inches between the main piles.

The crown of the south pier is 26 feet 3 inches wide, rounded on the upper surface, which attains the level of ordinary high water. The piles connecting the mattrasses are carried to a height of 9 feet 10 inches above this level. A timber roadway, carrying two lines of rails, is attached to the piles.

Open Timber Frames are very often employed for piers and wharfs where the water is tolerably quiescent and but moderately deep. The frames may be either fixed or movable. In the first instance, the verticals consist of whole timber piles, generally greenheart or creosoted pitch pine, driven down to a solid stratum and connected transversely above the water level by cross pieces and inclined struts, as at Hull (fig. 246). In the second case the verticals are tenoned into and rest upon a timber sole-plate, set upon a naturally hard bottom, as at Blyth (figs. 219 and 220). In both cases, the frames are erected at distances apart, usually



Figs. 219 and 220.—Jetties at Blyth.

from 10 to 15 feet, and the bays thus formed are faced with horizontal walings and fenderings. The movable frames have necessarily to be weighted down with heavy stone filling, and this is frequently added in the case of fixed frames, in order to stiffen the work. A foundation of concrete is occasionally to be found, as at Liverpool, and exemplified in three instances (figs. 221, 222, and 223), especially when it can be utilised in the formation of culverts with sluice openings to maintain the required depth of water in situations where there is a tendency to silting. A concrete apron must then be added to the structure, or it will inevitably be undermined by the current. Piled timber jetties have also been constructed upon a rock bottom. At Newcastle, for the uprights of coaling staiths, holes, 3 inches in diameter, were drilled into the rock and into these the pile shoes, which had 4-inch square spikes, 6 feet long at their ends, were driven. At Liverpool, similar but larger holes were drilled for the Prince's jetty, the holes being 25 inches diameter, and

Lower Clay

Fig. 221.—Jetty at Liverpool—Type A.

Lower Clay

Fig. 222.—Jetty at Liverpool—Type B.

consequently capable of receiving the whole butt ends of greenheart piles, 14 inches square, which were grouted in concrete after being adjusted.

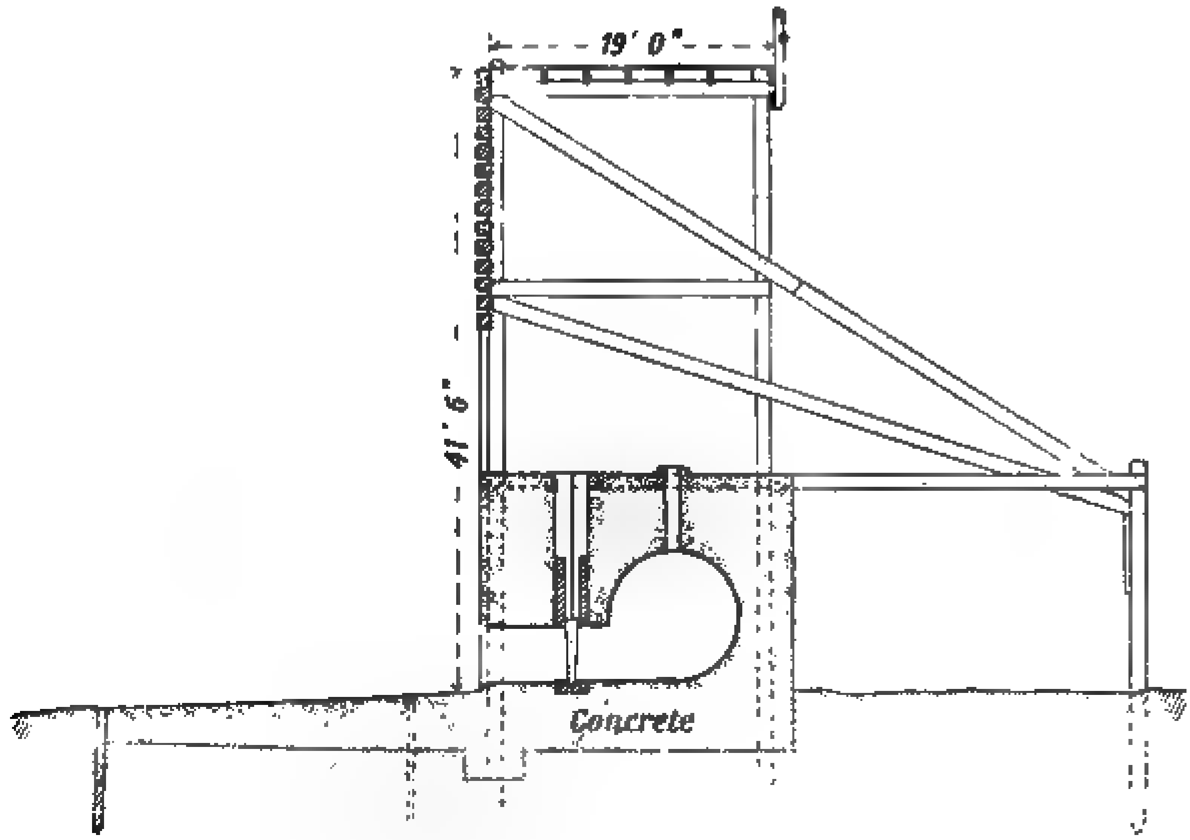


Fig. 223.—Jetty at Liverpool—Type C.

Crib Work is a mode of construction peculiarly characteristic of jetties in the large North American lakes. From the crudeness of its build and the perishability of the material, the system must be regarded as mainly of

Fig. 224.—Crib Frame.

the nature of a temporary structure; indeed, it is doubtful whether it is applicable to other than the particular localities in which it has been devised and practised, admittedly with success, where timber is plentiful and cheap, and where present requirements outweigh considerations of future contingencies. Cribs are box-shaped frames of timber (pine, cedar, ash, tamarac, or elm), constructed in open-work, with numerous compartments formed by means of transverse and longitudinal ties. They range from 30 to 50 feet in length and are never narrower than the total height, with a minimum in the shallowest cases of 20 feet. The main timbers are 12 inches square throughout, except in the lowermost course, or grillage, where they are 12 inches by 18 inches. The transverse and longitudinal ties are about 10 inches by 12 inches, and the structure is held firmly together by $1\frac{1}{8}$-inch wrought-iron bolts. This method of construction will be tolerably clear from an inspection of fig. 224.

The preparation of the site for the cribs is a matter of importance. A sandy bottom is not very suitable, giving rise to unequal settlement. A mound of rubble has been found to answer the purpose best.

The cribs are framed on a sheltered beach, within easy reach of a draught of 10 or 12 feet of water. After three or four courses have been bolted together the structure is launched, and additional courses put on until the height is several feet greater than the depth of the jetty site. The crib is then towed into position and weighted with stone until it sinks, after which it is filled level with the top. After the final settlement, all the cribs are levelled up with wedges, and a roadway of planking is laid at a height of 5 or 6 feet above water level. The cost of crib work in 24 feet of water at Chicago, in 1871, amounted to about £30 per lineal foot.

Iron Columnar Piers form light, ornamental structures, and they are often adopted where the traffic is mainly in passengers. The open columns also cause practically no interference with the movements of the sea, and consequently the type is a suitable one in situations where there is a littoral current which it is inadvisable to deflect in any way. The columns are either piles themselves or are bolted to the heads of piles, unless the bottom surface be rock, in which case there is no need for piling. Screw piles are very generally employed, on account of the broad bearing afforded by the surface of the screw. The columns are arranged in bays, and are connected just below the decking by longitudinal and transverse girders, the depth and design of which will depend upon the distance apart of the columns. There is so much scope for individual taste and opinion that it is impossible to lay down any rules, of a general nature, in regard to the design of iron columnar piers. Two examples will suffice by way of illustration.

At the port of Soukhoum, in the Black sea, there is an iron pier (figs. 225 and 226), about 154 feet long, constructed in 1889. The bays are each 14 feet in extent, with one of 7 feet at the end. There is also a further

Figs. 225 and 226.—Pier at Soukhoum.

PORTION ADJOINING THE SHORE

Fig. 227.—Jetty at Zeebrugge.

projection of 7 feet at the outer extremity, forming a support to a stairway. The columns are arranged in parallel rows of five, the middle columns being 7 feet 10½ inches apart and the outer ones 5 feet 3 inches. They are 5 inches in diameter, connected by 3-inch by 3-inch by ⅜-inch angle-iron bracing. The deck is planked upon whole timber bearers, at a height of 14 feet above the water level.

In order to allow freedom of movement to the littoral current a portion, 440 yards long, of the jetty or mole at Zeebrugge, on the North Sea, is constructed of mild steel in openwork. The structure (figs. 227 and 228) is composed of 80 bays of 16½ feet each, and is carried by parallel rows of piles or columns, six in number, of which four support a double line of rails and two the side extremities of the platform. The heads of the piles are connected by a lattice girder, and at low-water level, there is a second horizontal member formed of two channel irons. The diagonal bracing is 2 inches in diameter, fitted with tightening-up shackles. Each column is formed of four quadrant irons, rivetted together at their longitudinal flanges. The internal diameter is 9½ inches and the thickness ⅝ inch. The sectional area of each pile is about 31 square inches. At the foot of each pile is a wooden shoe, 16 inches in diameter and a yard long, bearing against a collar on the pile.

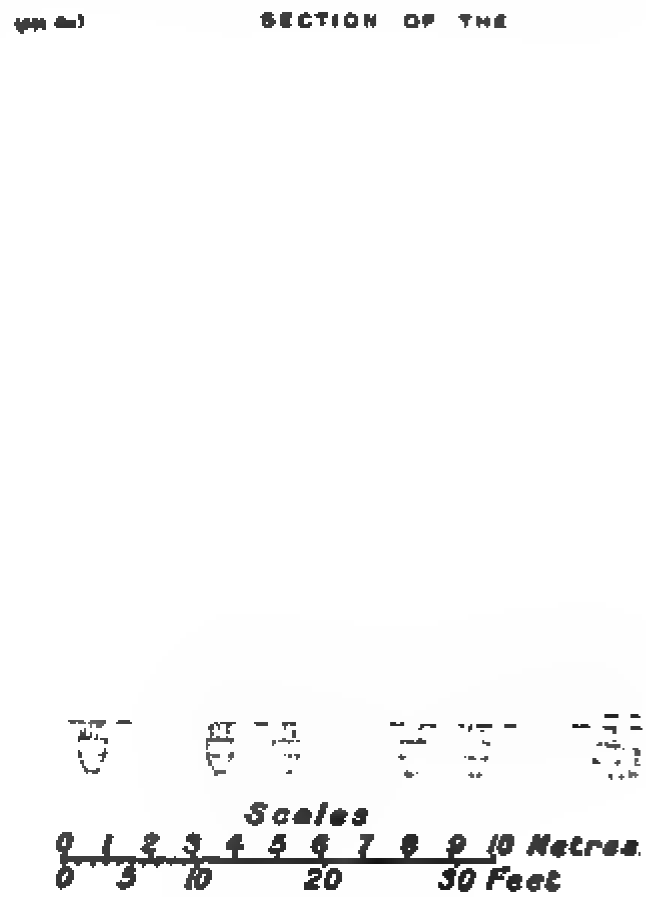


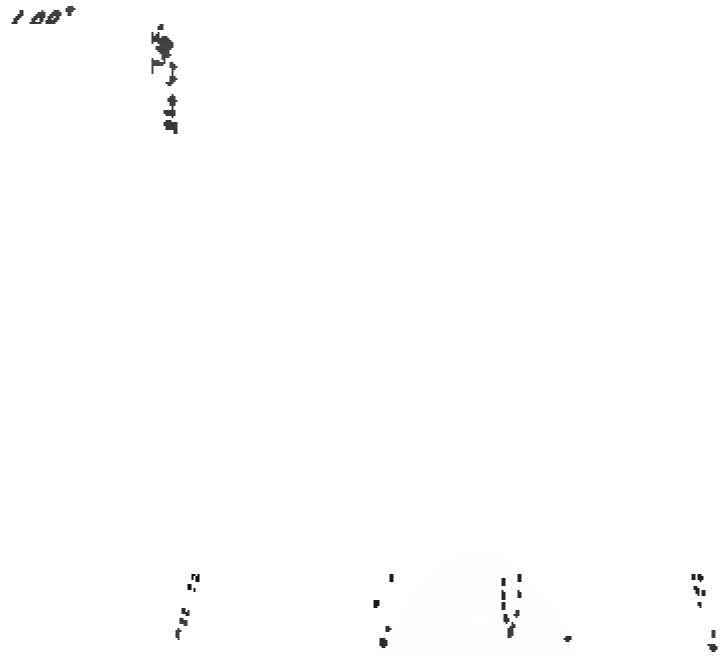
Fig. 228.—Jetty at Zeebrugge.

The rows of piles are connected longitudinally by plate girders, four of which are 2 feet 6 inches deep and the outer two 2 feet deep, with 6-inch by ½-inch flat-bar wind bracing. The decking comprises 5-inch by 2½-inch oak joists, set 1¼ inches apart clear, to allow a passage for waves. The cover-plates in the railways are of cast iron, pierced pattern.

On the outer face of the jetty, there is a plate superstructure, 15 feet 9 inches in height, suitably stiffened, to afford shelter to trains. This superstructure carries a gangway for pedestrians.

An interesting example of a jetty of a somewhat unusual type for iron, though not for wood, is given in figs. 229 and 230, which is a section of one constructed at the port of Touapsé, on the Black Sea, in 1896-97. The jetty is 800 feet long and has two inclined faces, each formed of a row of railway metals on end, driven into the ground some 10 inches apart, being guided and strengthened by two rows of longitudinals; the upper, 9 feet

above the water line, an angle iron 6 inches by 5 inches by ½ inch, and the lower, a channel iron 10 inches by 3 inches by ½ inch, connected in each case by transverse through-bolts. The interior of the jetty is filled with rubble. The top width is 14 feet, providing accommodation for a single line of rails.



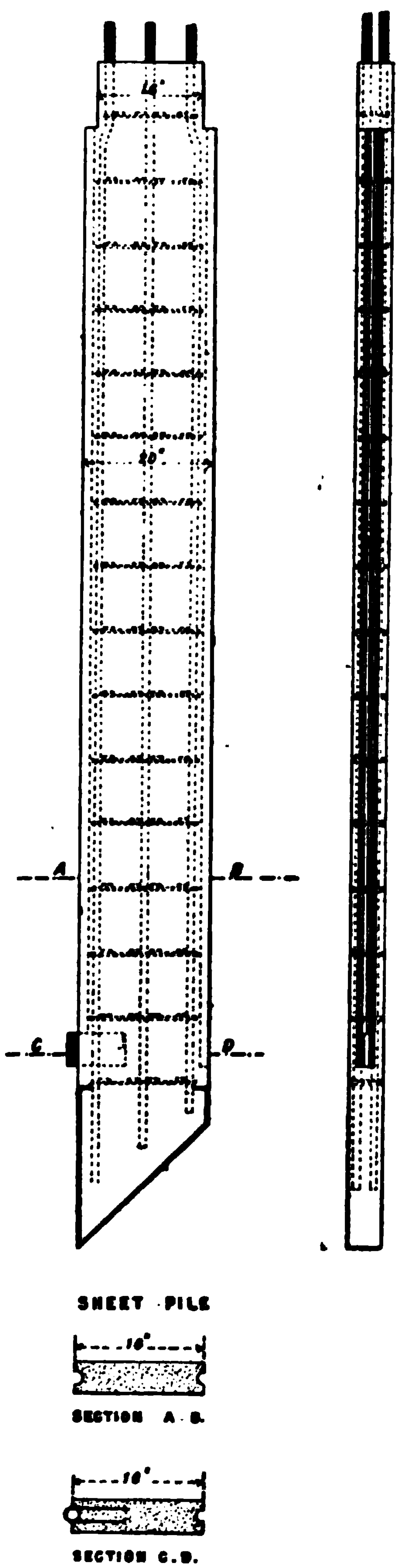
Figs. 229 and 230.—Jetty at Touapsé.

Composite Systems.—Perhaps the most remarkable development of recent years is the intimate combination of iron and concrete for building purposes, and not the least important application of the method is in reference to piling. The earlier open-work systems, whether of iron or wood, are subject to deterioration and decay—in the first case from corrosion, and in the second from the ravages of sea worms. Hence a combination of two materials, in which the durability of the one acts as a preservative to the strength of the other, is an undoubted advantage. Such is the principle of several well-known systems, in all of which iron rods and bars are completely imbedded in concrete, so as to be beyond the reach of external destructive agencies. Two of these systems, from their primary application to building construction, are more fully described in the chapter on Sheds and Warehouses. Here we are only concerned with their adaptability to jetties and piers.

The *Monier system*, consisting of a mesh of metal wire incorporated in a slab of concrete, has been used as an external cover for timber piles, and also, in the form of cylinders, for bridge foundations. Monier tubes, in 3 feet 6 inch lengths, 21 inches internal diameter, 1⅜ inch thick, with a hearting of steel wire netting, 1¼ inch mesh, No. 16 gauge wire, have been used by Mr. De Burgh for the protection of ironbark piles in Australia. In a second instance, the cylinders were 3 feet 6 inches in diameter, with a thickness of 2½ inches. Both applications were successful, and indicate the possibility of utilising Monier tubes on a more extended scale for marine foundation work.

The *Hennebique system* has been more directly applied to the formation of piles. As practised in recent examples, it consists in enclosing rows of iron rods, bound at intervals by iron ties, in a casing of concrete. Figs. 231 to 234 are the elevation and sections of a sheet pile, constructed in this manner. There are three rows of pairs of vertical rods, connected, at 10-inch intervals, by horizontal bands or clips. The pile is moulded with cylindrical grooves in each side, in which the spur, C, of an adjoining pile engages, for guidance in driving. When two consecutive piles have been driven, their combined grooves form a cylinder, which, after being cleansed by forcing water through it under pressure, is grouted with cement. The lower ends of the piles, which can be made either wedge-shaped or pointed, are protected by steel shoes secured to the body of the pile in the moulding process.

In fig. 27 (p. 63) is a plan showing the method adopted for the construction of bearing piles. Piles of this description, 14 inches square and 42 feet long, have been driven to the number of 1,300 for a cold storage foundation at Southampton. A monkey weighing $2\frac{1}{2}$ tons was used, and the piles were driven until 10 blows, with 4 feet 6 inches fall, failed to produce an additional inch of depression. It is better for this class of work to use a heavy weight with a short fall, rather than a light weight with a long fall. Owing to the brittle nature of the concrete, the head of the pile during driving must be protected, as shown in figs. 235 and 236, by a sheet helmet bedded on sawdust or sand in bags on the head of the pile, with the further interposition of a wooden dolly between the monkey and the helmet. The loss of energy by this arrangement is very great, though eventually the sawdust hardens into a compact mass.

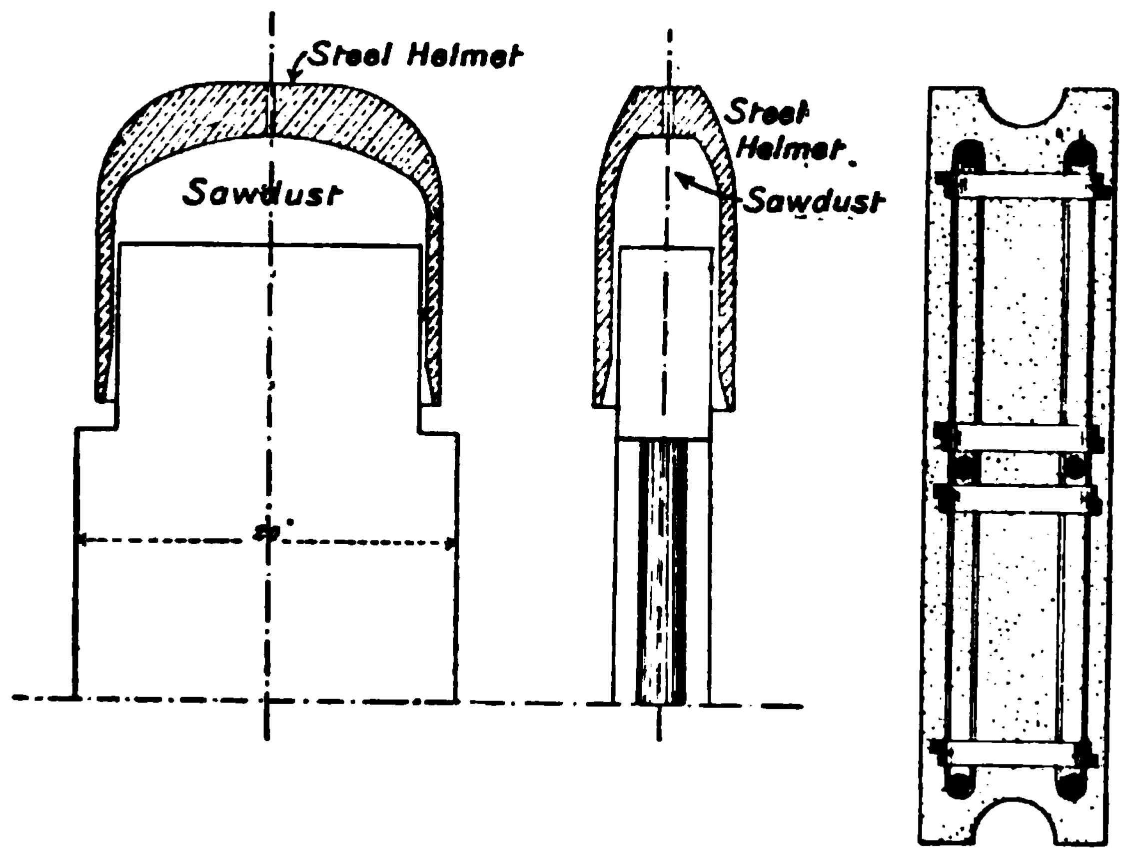


Figs. 231, 232, 233, and 234. Hennebique Sheeting Pile.

The brittleness and rigidity of the thin concrete covering are the only drawbacks of the composite system in positions such as jetties, where it is liable to concussions and shocks. It has, however, been used with satisfactory

results in the construction of a jetty at Woolston, near Southampton, and possibly in other cases.

HENNEBIQUE SHEETING PILES



Figs. 235, 236, and 237.

Jetties and Wharfs at Belfast.*

Fronting the Victoria Channel and flanking the entrance to the Alexandra Graving Dock at Belfast, two timber jetty quays or wharfs have been erected, the former 510 feet long and the latter 840 feet. The structures comprise eight jetties, connected by a narrow wharf (fig. 238), extending

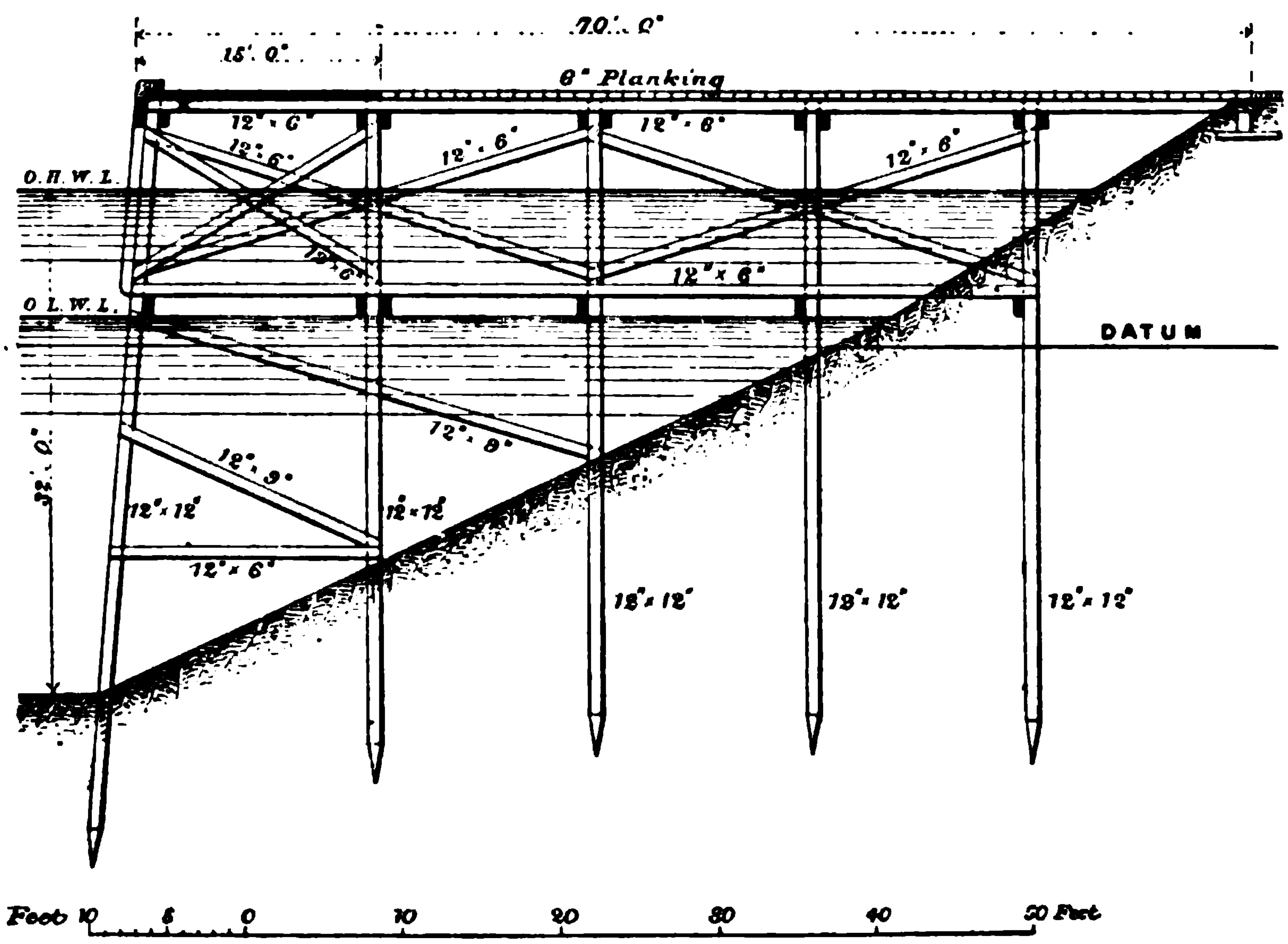


Fig. 238.—Wharf at Belfast.

* Kelly on "The Alexandra Graving Dock, Belfast," *Min. Proc. Inst. C.E.*, vol. cxi.

70 feet from the quay face. The jetties are 20 feet and the wharf 15 feet wide. All the timber work is of pitchpine except the fenders, which are of American rock elm. The coping timber is protected by a sheathing of malleable iron, $2\frac{1}{2}$ feet broad and $\frac{1}{4}$ inch thick. The pitchpine was creosoted with 4 lbs. of creosote per cubic foot, which, owing to the density of the timber, was with difficulty forced into it. Twenty-five mooring bollard piles, 48 feet long, were driven and secured to the main framing of the jetties and quays, and fitted on the top with cast-iron bollard caps. The bays of piling of the jetties are 9 feet apart, and of the quays between the jetties, 8 feet. The slopes behind the jetty quays down to the lowest low-water mark are protected with 16-inch rubble whinstone pitching, laid on a 6-inch bed of gravel.

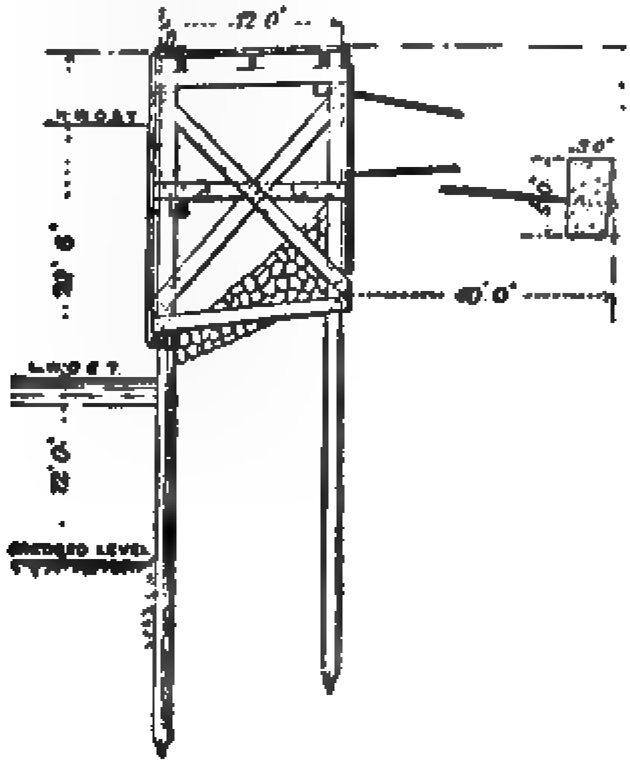
A quay wall, 80 feet in length, and three piers and foundations for supporting a 100-ton derrick crane were constructed of concrete in the tide-way, within two rows of sheeting piles, 38 feet long. The three crane piers are raised 22 feet above the quay level; they are 20 feet square at the base and $17\frac{1}{2}$ feet square at the top, and have wrought-iron holding-down bolts. Plate castings are built into the piers for securing the granite seats and the foundation castings of the crane. Along both sides of the derrick crane seat, a timber wharf, very economical and serviceable in form where the depth of water in front of the quay is not greater than about 10 feet at ordinary low water, was constructed for a total length of 220 feet. Forty-five bays of supporting piles, in front, and stay piles, four in each bay, behind, were driven, 5 feet apart, along the wharf. A row of sheeting piles, 22 feet long and 7 inches thick, was driven along the face of the wharf. For a depth of 14 feet below the coping level, close 4-inch planking was spiked to the back of the front row of supporting piles, and a coping timber, 18 inches by 10 inches, was secured along the quay face to the pile-heads. The wharfing was tied back by iron bolts to the stay piles, and the space immediately behind the face-work was filled in with ashes, brick rubbish, &c. The cost of such a wharf facing amounts to between £10 and £12 per lineal foot of frontage.

Timber Wharf at Dundee.*

The landing wharf (figs. 239 and 240) at present in use for the discharge of steamers, and available at any time of the tide for vessels whose draught is too great to admit of entrance into the docks, has a length of 2,800 feet, and is provided with shed accommodation at the rear to the extent of 24,650 square yards. It is 12 feet 6 inches in width, is constructed of two rows of main piles, 9 feet apart centre to centre, with sheeting between the piles of the first row, and is tied back by iron tie-rods $\frac{3}{4}$ inch diameter and 50 feet long. There are bollards along the face of the wharf, 18 feet apart, and numerous ladders down to low-water level. The timber wharf being of

* Buchanan on "The Port of Dundee," *Min. Proc. Inst. C.E.*, vol. cxlix.

a temporary nature, and more shed accommodation being required, the construction of a permanent river wall, 140 feet outside the present wharf, is about to be commenced.



Figs. 239 and 240.—Wharf at Dundee.

Jetty at Dunkirk.*

The new east jetty at Dunkirk (fig. 241) has a foundation of masonry, constructed on the compressed air system described in Chapter v. The

Fig. 241.—Jetty at Dunkirk.

* Barbé on "Travaux les plus récents executés dans les principaux ports français," *Eighth Int. Nav. Cong.*, Paris, 1900.

jetty is 938 yards long, and 42 caissons were employed, generally 68 feet long, with widths ranging from 15 feet 6 inches to 21 feet for the jetty and 31 feet for the pierhead. These caissons were sunk into the fine sand of the beach to depths of 16 to 26 feet below zero. The joints between the caissons were at the most 20 inches wide: they were simply closed by wood panels. The superstructure consists of an open timber framework adjoining the mainland, 650 feet long, a half-filled-in framework, 490 feet long, and a solid breakwater for the remaining length, bordered on the inner side by a stockade and on the outside by a mole, having a hearting of sand protected by a facing of masonry. In the open-work jetty the masonry base is carried to a level of 8 feet above datum and to a level of 16 feet 6 inches in the half solid portion. The platform is constructed throughout at a height of 29 feet 6 inches above zero, or 6 feet above high water of equinoctial spring tides.

River Jetties at Tilbury.*

The jetties at the entrance to the tidal basin are 45 feet in width, and project in the tideway into about 45 feet of water at high water of spring tides, or 48 feet below Trinity high-water mark. The centres of the rounded ends of the jetties were formed of cast-iron cylinders, 15 feet in diameter, sunk to a chalk foundation at about 75 feet below the level of the deck. These cylinders were afterwards filled with concrete. Immediately around the cylinders, and hooped at intervals to them, was driven a double row of piles, from which radial and cross strutting was carried to the outer piles. The straight portions of the jetty were formed by a double row of piles on each side in 10-foot bays, with four horizontal struts, and cross strutting extending the full width of the structure. The whole of the piles and main timbers were of sawn pitchpine logs, averaging about 14½ inches square and 65 feet in length. The decks were formed of 3-inch planking in 4½-inch widths, laid upon 11-inch by 2-inch bearers. The shore end of the west jetty was similar in construction to the outer ends, and the corresponding end of the east jetty was connected with the solid knuckle formed by the return of the south wall of the tidal basin.

Pierhead at Madras.†

The pierheads at Madras Harbour are formed of cylindrical monoliths, consisting of a plating of iron with a concrete interior (figs. 242 and 243).

For each pierhead a watertight iron caisson was provided, with outside diameters of 42 feet and 41 feet 5½ inches at the base and summit respectively, and 53 feet in height. The bottom and sides were covered with ¼-inch plating, the latter being built up in a series of tiers or horizontal

* Scott on "The Tilbury Docks," *Min. Proc. Inst. C.E.*, vol. cxx.

† Thompson on "The Caisson at the North Pierhead, Madras Harbour," *Min. Proc. Inst. C.E.*, vol. cxxv.

bands, each consisting of eight curved plates, 16 feet 6 inches long and 4 feet in height. Both sides and bottom were strengthened with ribs of lattice girders. Across the bottom, each along the centre of a row of plates, ten girders, 2 feet in height, were placed, 3 feet $9\frac{3}{16}$ inches apart from centre to centre. The sides were supported by fifteen circular girders, placed horizontally, and varying in breadth from 1 foot 9 inches to 1 foot 6 inches, and also by twelve vertical girders from 2 feet to 1 foot 9 inches

Scale, 24 feet to 1 inch.

Figs. 242 and 243.—Pierhead at Madras.

in width. The vertical girders were set at equal distances apart, and only their inner flanges were continuous throughout the height of the caisson, the outer flanges being arranged in sections between the horizontal girders. The inner flange consisted of a 3-inch by 3-inch by $\frac{3}{8}$-inch angle iron, connected at a single joint by a bar cover. To these girders the side plating was fixed by $\frac{3}{4}$-inch rivets at 5-inch pitch, the tiers being rivetted together with $\frac{5}{8}$-inch rivets at $2\frac{1}{2}$-inch pitch and arranged telescopically,

so that each tier is $\frac{1}{2}$ inch less in diameter than that immediately below it.

The caisson was furnished with four 12-inch sluice valves, fitted to the outside of the eighth tier, 27 feet 7 inches from the bottom. Eighteen 3-inch wrought-iron pipes, rivetted over 3-inch holes in the bottom, were also provided for the purpose of grouting the rubble base beneath the caisson. They were built to a height of 50 feet, in three lengths, with screw ends. This great height was necessary, as the base could not be grouted until the caisson was nearly filled with concrete, but it entailed considerable difficulty in affixing successive lengths, a step which had to be undertaken while the caisson was afloat and by no means quiescent. Being too slender to support themselves, they had to be stiffened by bracing to the sides of the caisson, an arrangement which interfered with the lowering of material and plant. Any damage, moreover, to the pipes below the water line would, in all probability, have involved the foundering of the caisson.

The caissons for the north and south pierheads were similar in construction, with the exception that the former had an additional bracing of three transverse bottom girders, 2 feet deep, rivetted over the tops of the other ten, at right angles to them. The north caisson was brought over from England in sections and put together, to a height of 23 feet, within a temporary dock or enclosure on the beach. At this stage it was launched, and received a solid floor of concrete 4 feet thick. Above this floor, concrete was deposited, to a height of 3 feet, in such a manner as to leave seven circular wells or pits, which, with the exception of the centre one, used as a tide gauge-well, were filled later. The lining for these and the sides was built by means of wooden moulds, 5 feet 6 inches in height, set upon wooden putlogs as the sides were raised. When the iron sides with their concrete lining were completed the caisson drew 36 feet of water. At this draught it was berthed over the site, which had a prepared rubble foundation, with a slight inclination, to cause the caisson to tilt slightly inwards towards the blockwork of the pierhead and the wave-breakers, which would lean against it. When in position, the sluice valves were opened, the caisson grounded, and about 500 tons of water were admitted, sufficient to keep it secure. The pits were then utilised for the reception of a number of concrete blocks, ranging from 25 to 150 tons in weight, and the caisson was subsequently emptied of water by a pulsometer. After this, the work of completing the concrete interior was proceeded with without interruption. The sluices were removed at the close of the work.

Piers at Sunderland Harbour.

These piers consist of two curved arms, projecting from the shore line and converging to a distance apart of 480 feet at the pierheads. The area thus enclosed is 100 acres.

The Roker Pier (fig. 244), on the north side of the River Wear, has a

length of 2,800 feet. For 2,340 feet of this length, the width at the top is 35 feet, while for the remaining portion, the width is 41 feet. The width at the bottom varies with the depth, and is generally 120 feet at a depth of 40 feet below low water. The top of the pier is 10 feet above high water. A subway, 6¼ feet high by 4 feet wide, runs the entire length of the pier, and affords access to the lighthouse in stormy weather. The shoreward portion of this pier, for a length of 385 feet, is constructed of concrete *en masse*, faced with granite blocks; for the remainder of the pier, the superstructure is formed of granite-faced concrete blocks, varying in weight from 43 to 54 tons, set in lengths of 42 feet 7 inches each, by a radial hydraulic block-setting crane, which could set a 60-ton block 60 feet in advance of its leading wheel. The interior of each length is filled with concrete blocks and mass concrete. The superstructure is set on a foundation levelled to 2½ feet above low water. This foundation was formed of 56-ton and 116-ton bags of 4 to 1 concrete deposited in a plastic condition on the rock. The concrete was enclosed in bags of jute sacking, weighing 27 ounces per yard, 30 inches wide. These bags were made in boxes slung in the well of a Wake twin-screw bag-barge and suspended from hydraulic cylinders. The

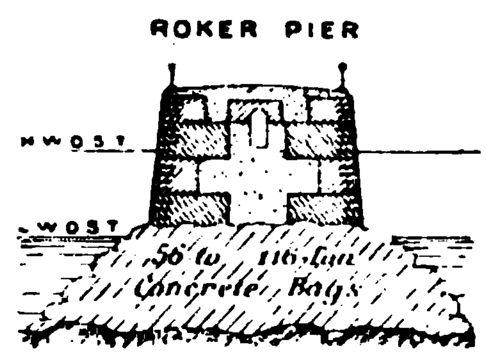


Fig. 244.—Pier at Sunderland.

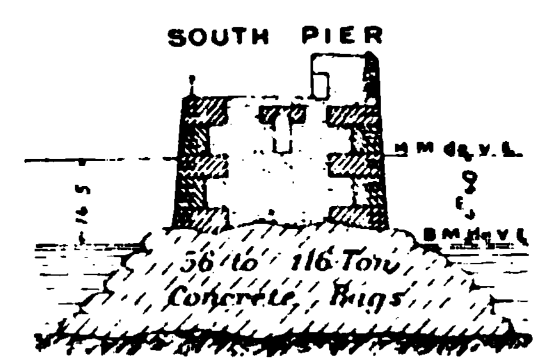


Fig. 245.—Pier at Sunderland.

barge steamed alongside a concrete mixing-house, where the bag was filled with plastic concrete and laced; the barge then proceeded to sea and was moored directly over the place where the bag was required. The box and bag were then lowered as near the bottom as possible and the bag deposited. For a length of 460 feet at the outer end of the pier, the rock was covered with a layer of sand, varying in thickness from 1 to 17 feet, and this was removed by a sand pump dredger before the bags were deposited.

The pierhead is formed, in the first place, of an iron caisson, 100½ feet long, 69 feet wide, and 26½ feet deep, set on a specially prepared foundation of concrete bags, levelled to 23 feet below low water. The caisson was floated out with a draught of 22 feet, containing 3,500 tons of concrete, and sunk on its site by partly filling it with water. It was then built up with 15-ton and 25-ton blocks, mass concrete and cement-grouted granite rubble until, when completed, its weight amounted to 10,000 tons. On top of this the pierhead superstructure was constructed in blockwork and surmounted by a lighthouse, giving a total weight of 23,000 tons for the whole structure.

The new south pier (fig. 245), on the south side of the harbour is constructed in a similar manner to the Roker Pier, but varies somewhat in

details. The length of the pier is 2,844 feet, the width is 35 feet for three-fourths of this length and 41 feet for the remainder. The top of the pier is 9 feet above high water, and there is a parapet wall, 9 feet high by 9 feet wide, mainly, but 14 feet wide at the outer end, running along its entire length. The weight of the blocks used on this pier was 15 tons; they were set on a bagwork foundation by a 20-ton block-setting crane worked by a gas engine. The crane revolved completely, and could set a 20-ton block 64 feet in advance of its leading wheel. The foundation was constructed in the same manner as that at Roker Pier.

Wharfs at Greenock.

The wharfs constructed along the frontage of the River Clyde, at Greenock,* between the entrances to the East and West Harbours and westward of the West Harbour, in order to obtain a greater depth of water than existed at the old quays, are known as the Steamboat Quay and the West Quay respectively. These wharfs were erected parallel to and 25 feet back from an improved channel way, adding about 5,380 superficial yards to the old irregular quays—which are much used for coasting traffic—and a depth of 28 feet at high water has been provided in front of them. Borings, taken along the line of the new work, showed that a firm stratum, fit for quay wall foundations, was only reached at great depths, attaining 70 feet below high water in one or two places, and therefore timber work was adopted. A trench was first dredged along the front line of the new work, and, after driving the piles, a bank of whinstone rubble was deposited, to serve as a toe to the filling between the new and the old work. To increase the resistance of the main piles to outward thrust, wrought-iron shields, 5 feet by 3½ feet, were bolted to the faces of the front piles before driving, and then driven down so that their tops were 2½ feet below the level of the finished dredged bottom. Sheeting piles and horizontal planking were placed along the line of the front piles to retain the bank of rubble stone, and for the retention of the filling behind the back line of main piles, a double row of sheeting piles was driven, the lower ends of which extended about 4 feet into the rubble bank, and between the sheeting piles, a wall of 8 to 1 concrete was brought up to the deck planking. The greenheart front and back piles, 14 to 16 inches square, 8 feet apart, and driven into the hard clay, are joined by half-timber ties, and whole-timber struts were inserted between the piles, and the ties and struts bolted together. The quay surface is planked with 3-inch Gardnerised fir planking, with whinstone pitching laid thereon, on a bed of Portland cement mortar. The face of the quay is protected by segmental rubbing irons.

* Kinipple on "Greenock Harbour," *Min. Proc. Inst. C.E.*, vol. cxxx.

Wharfs at Hull.

The splayed wings of the entrance to the Alexandra Lock, at Hull,* are lined with timber wharfs, which are returned for a length of 300 feet up and down the River Humber. The wharfs (fig. 246) were constructed in bays, generally 10 feet in length, but 3 feet at the corners, the framing being braced both longitudinally and transversely, and covered with a 5-inch decking. The river bed in front of the wharfs had been dredged away to about 40 feet below the top of the piles, so that the piles, which were 61 feet in length and about 15 inches square, penetrated only about

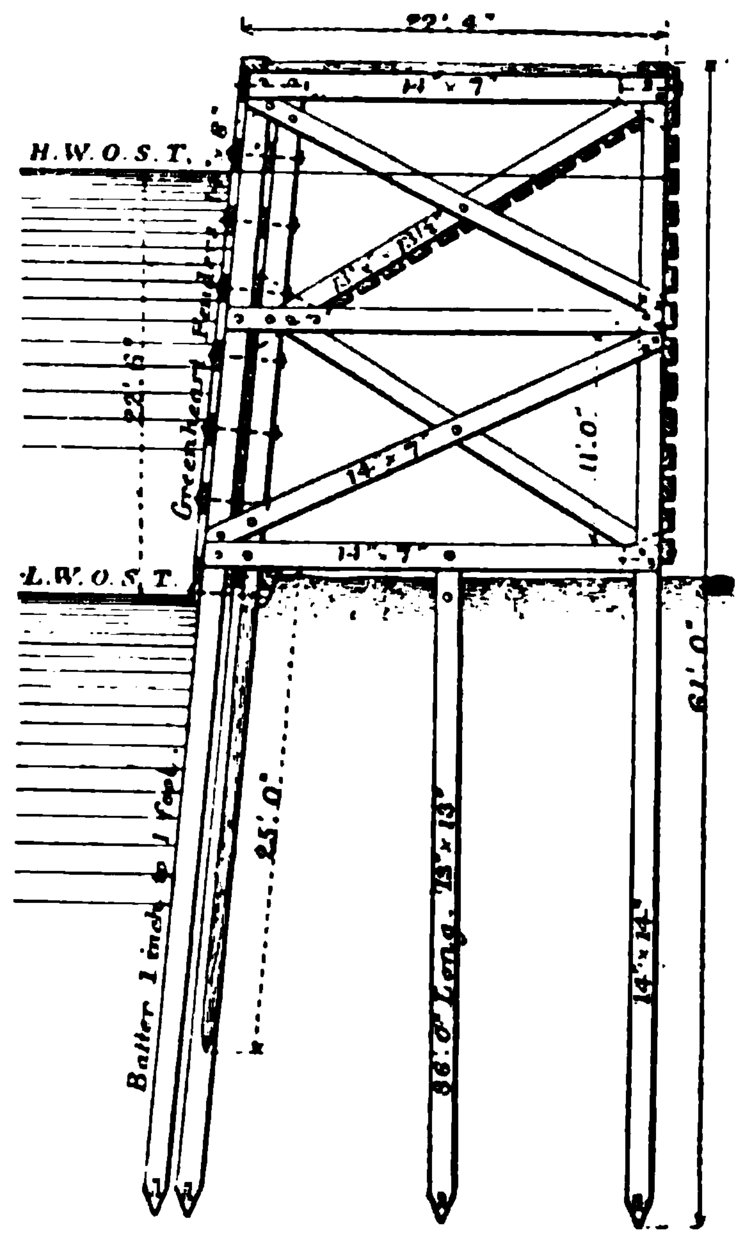


Fig. 246.—Wharf at Hull.

20 feet into the ground. Grooved and tongued sheet piling, 25 feet long and 8 inches thick, was driven along the front, the top being just above low water. The sheeting was driven in lengths of 6 feet at a time, all the piles in one bay being previously pitched in position so as to ensure tight contact. This sheeting held up the material at the back when the river bed was deepened in front. During construction the mud accumulated so rapidly, in the recesses behind, that whole-timber sheeting had to be driven at the back to retain it, the space enclosed between the front and back piles being excavated to enable the cross bracing to be fixed at the lowest possible level. The wharfing was constructed from a staging on piles driven by piling machines on barges. The sheet piling was driven by piling machines with telescopic leaders.

* Hurtzig on "The Alexandra Dock, Hull," *Min. Proc. Inst. C.E.*, vol. xcii.

CHAPTER VIII.

DOCK GATES AND CAISSONS.

DEFINITION AND RELATIVE ADVANTAGES OF GATES AND CAISSONS—METAL *versus* WOODEN GATES—WEIGHT, COST, DURABILITY, AND STRENGTH—SINGLE-LEAF AND DOUBLE-LEAF GATES—HORIZONTAL AND VERTICAL GIRDER TYPES—STORM GATES—STRUT GATES—STRESSES IN GATES—STATICAL FORCES—METHODS OF FINDING RESULTANT PRESSURE—ZONES OF EQUAL PRESSURE—RISE OF GATES—ANALYSIS OF RESULTANT—GRAPHIC REPRESENTATION—LIMITS OF STRESS—TYPICAL EXAMPLES—VERTICAL CO-PLANAR GIRDERS—STRESS IN PANELS—EXEMPLIFICATION OF GATE CALCULATIONS—FITTINGS—EXAMPLES OF GATES AT LIVERPOOL, BIRKENHEAD, MANCHESTER, HULL, BUENOS AYRES, CALCUTTA, SOUTH SHIELDS, AND DUNKIRK—TABLE OF DOCK GATES—STRESSES IN CAISSONS—CLASSIFICATION OF CAISSONS—SWINGING, TRAVERSING, SLIDING, ROLLING, FLOATING, AND SHIP CAISSONS—LOWERING PLATFORMS—EXAMPLES OF CAISSONS AT MALTA, BRUGES, BLACKWALL, CARDIFF, CALCUTTA, BELFAST, LIVERPOOL, AND GREENOCK—TABLE OF DOCK CAISSONS.

IN localities where there is considerable tidal range and where circumstances render it desirable to maintain the surface of the water set apart for the reception of shipping at a fairly constant level, it is evident that the entrance or entrances to a dock must be closed in order to impound the water, and must remain closed during those portions of each day in which the tide falls below a certain limit. This is usually effected by means of (1) gates or (2) caissons, and occasionally provision may be found for both forms of closure. Graving and repairing docks are treated in like manner, but for a different purpose, the object in this case being to exclude the external water during the time of occupancy.

Definitions.—The distinctive feature of a gate is that it revolves about an axis, in most cases vertical, but occasionally horizontal, while the motion of a caisson is either rectilinear or altogether untrammelled. As with many other terms, however, employed in dock engineering, this definition is not susceptible of too rigid interpretation. There is an intermediate class of gate-caissons combining the hinge or axis of the gate with the broad beam of the caisson, and exemplified at Bristol, Dundee, Havre, and other places, though, taken on the whole, the type is rare.

The Relative Merits of Caissons and Gates, considered as two distinct, though comprehensive, classes based on the foregoing definitions, may be broadly gauged as follows :—

1. Gates with vertical axes need side recesses into which they may be swung when the entrance is to be opened for the passage of vessels. This

necessitates a considerable and expensive addition to the length of the side walls, especially when the lock or entrance is of great width, as often obtains at the present day. Caissons do not occasion any increase in the length of the side walls, but, on the other hand, there must be reckoned the cost of a special chamber for sliding and rolling caissons. Ship caissons do not need a chamber, but, when out of use, they have to be berthed somewhere, and this leads to a certain amount of inconvenience in the appropriation of useful space.

2. Caissons are generally of stronger build and broader beam than gates, and they afford accommodation for the transmission of rail and road traffic across a waterway, thus discharging the functions of a bridge in addition to those peculiarly their own.

3. The first cost of a caisson is undoubtedly, in most cases, greater than that of a pair of gates, but if the cost of a swing bridge for vehicular traffic, which is a necessary adjunct in the case of gates, be also taken into consideration, the advantage will be found to lie with the caisson. This advantage is still further emphasised where a lock or passage is fitted with double gates to alternately impound or exclude water. A caisson can be constructed to act equally in both directions.

4. Caissons obviate the necessity for pointed sills and gate platforms of large area, but those of the ship type, fitting into grooves so as to be capable of acting in two directions, call for battered side walls to allow of their floating clear when manœuvring in and out of position, and this gives the entrance an unsuitable profile for modern vessels of square amidship section and with bilge keels.

5. Floating caissons are not always manageable in boisterous weather and strong currents, and oftentimes they are only workable with difficulty. Sliding caissons, too, have to encounter the effect of wind pressure, especially if there be much clearance between their keels and the sliding ways. Neither can rolling caissons be said to be altogether exempt from the abrading or wearing effect due to the action of friction on the moving parts under lateral pressure. So that, on the whole, it may be claimed that gates are easier of movement and are more completely under control during manipulation.

Dock Gates.

Gates are sometimes distinguished as wooden gates or iron (including steel) gates, according to the nature of the bulk of the material of which they are composed. As a matter of fact, both materials enter essentially, though in varying proportions, into the construction of all gates. It would be impossible to connect the various members of a wooden gate without the aid of metal bolts, straps, and other fastenings, while iron gates depend for their watertightness (except in rare instances) on wooden posts and plates at the abutting surfaces.

As regards the relative advantages of wood *versus* iron gates, the following points may be noted :—

1. **Dead Weight.**—For a given width of entrance, wooden gates are considerably the heavier. Greenheart is the wood now most extensively adopted in this country, but in spite of the fact that its specific gravity, though high for timber, is considerably less than that of either wrought iron or steel, being only 1·1 to 1·2 as compared with 7·6 and 7·8 for the metals respectively, yet it outweighs them both by reason of the excessive bulk required to offer an equal resistance to stress. This disparity in strength is still further emphasised in the case of the lighter woods, such as oak and pitch pine, considerably in vogue at Continental ports. And it must also be observed that no inconsiderable addition is made to the weight of a pair of gates by the unavoidably extensive use of metal fittings and connections. The weight of a pair of iron gates, 25½ feet deep, at Dublin, for a 70-foot entrance is stated to be 90 tons. A similar pair of steel gates at Limerick are about the same weight, while a 69-foot lock at Dunkirk possesses iron gates, 24 feet deep, weighing 88 tons. As against these fairly representative values for metal gates may be set the weight, 204 tons, of the wooden gates (48 feet deep) to a 70-foot lock at Avonmouth. These gates are mainly framed in pitch pine and memel, the heelposts and mitreposts alone being of greenheart. The weight of the iron fittings, including a cast-iron roller path, amounts to 42 tons. At the south lock of Buenos Ayres Harbour, the waterway is 5 feet less in width and 13 feet less in depth, but the gates weigh as much as 206 tons, owing to their entire construction in greenheart. For entrances of greater width, wooden gates attain enormous figures, as, for example, the greenheart gates (44 feet deep) at a 90-foot passage at Liverpool, which weigh no less than 330 tons. It is quite safe to assert that a pair of metal gates of the same size would not exceed half that amount.

2. **Effective Weight.**—Not only is the dead weight of wooden gates necessarily much in excess of that of iron gates, but the practicability of forming watertight compartments in the latter, constitutes a means of still further reducing the actual working load, since the flotation power thus obtained may be arranged so as to practically counterbalance the weight of the gates, leaving only a small margin for stability. By this means the power required for opening and closing the gates is reduced to a minimum. Even in localities where there is very great tidal range, and where anything like an exact counterbalance would be attended with much difficulty and some danger, the reduction in weight which can be safely made is far from negligible. At Dunkirk there were, some short time back, two similar entrances, 69 feet wide, one fitted with iron and the other with wooden gates. When immersed at mean sea level, the weight of the iron gates was reduced from 98 to 16 tons, to which 16 tons of water ballast was added making 32 tons in all. The wooden gates, when immersed, weighed just double this last amount. They have now been replaced by iron gates.

3. **Initial Cost.**—Generally speaking, gate materials may be placed as regards cost in the following order, commencing with the most expensive:—Greenheart, iron, oak, and creosoted pine. The exact proportion, of course, depends on current prices. At the present time, greenheart logs of large size can hardly be obtained for less than 3s. 6d. to 4s. per cubic foot, and for great lengths, the price will run as high as 5s. or 6s. Under such circumstances, greenheart gates, for entrances ranging between 60 and 100 feet in width, may be expected to cost, under normal conditions, from 40s. to 50s. per superficial foot of gate. Oak may be priced in this country at 3s. to 4s. 6d.; red pine at 2s. 3d. to 3s. 3d.; and pitch pine at 1s. 3d. to 2s. 3d. per cubic foot. Gates of these last named timbers will be relatively cheaper with a corresponding decrease in durability and strength. The cost of iron gates has fluctuated somewhat. In 1857 the Dublin graving dock gates cost 46s. 9d. per square foot of gate area, but the figure is a high one, and due, no doubt, to special and, possibly, local circumstances. The price of iron was certainly inordinately high about the year 1873, for the original intention of fitting the Avonmouth Lock with iron gates was abandoned in favour of wooden gates for that very reason. Iron gates constructed at Antwerp in 1873-74 cost 46s. 10d. per square foot. But in 1879, when estimates were obtained for a pair of gates at Dunkirk, the tender for ungalvanised iron had fallen to 21s. per square foot, and for galvanised iron it was only 26s. per square foot, including in both cases four coats of paint. About the same period Mr. Harrison Hayter, Past Pres. Inst. C.E., stated in the course of a discussion,* that he was in the habit of estimating the cost of wrought-iron gates at from 30s. to 40s. per square foot. Within the succeeding decade a pair of steel gates was erected at Limerick Dock entrance for 25s. 4d. per square foot. At the present time, allowing for market fluctuations, a pair of iron or steel gates might be expected to cost from 25s. to 30s. per square foot, with a slight margin in favour of steel.

On the Manchester Ship Canal, two pairs of gates were recently constructed for the same lock—one pair of greenheart and the other of steel. A statement (Table xxiv.) of their actual cost will be useful, if only as affording a basis of comparison between the two materials.†

From particulars of the cost of seventeen gates of oak for small entrances at German seaports, ranging between 25 and 45 feet in width, Messrs. Brandt and Hotopp have deduced 15s. per square foot as the average cost of such gates.‡ They further state that "the proportion in the cost of wooden gates to that of iron or steel gates may, under present conditions, be taken as 4 : 5, within the limits fixed for comparison."

* *Min. Proc. Inst. C.E.*, vol. lv., p. 72.

† Hunter on "Lock Gates of Greenheart and Steel," *Min. Proc. Ninth Int. Nav. Cong.*, Düsseldorf, 1902.

‡ Brandt and Hotopp on "Iron, Steel, and Wooden Gates," *Min. Proc. Ninth Int. Nav. Cong.*, Düsseldorf, 1902.

From which the cost of small metal gates in Germany may be considered as about 19s. per square foot—a figure very much lower than that quoted from Mr. Hunter's report, but some allowance must be made for the *locale* of the statistics, as well as for the difference in size of the gates.

TABLE XXIV.

Cost of Construction and of Erection of One Pair of Gates for a Lock, 65 *feet in width, with* 40 *feet of water over sill, exclusive of Operating Machinery and of Chains.*

Greenheart Gates.		Steel Gates.	
Timber,	£4,642	Steel and iron work,	£4,523
Iron and steel work,	1,604	Pumps and valves,	183
Labour,	1,640	Sheaves, &c.,	85
Erection,	603	Greenheart posts and sills,	425
		Pitch-pine fenders,	200
		Ballast,	206
		Erection,	138
Total,	£8,489	Total,	£5,760
i.e., 49s. 9d. per square foot of gate, or 65s. 3d. ,, waterway.		*i.e.*, 33s. 9d. per square foot of gate, or 44s. 3d. ,, waterway.	

Area of waterway = width of lock (65 feet) × greatest depth of water on sill (40 feet).

Mr. Nelemans states that, for a lock 40 feet to 60 feet in width, the cost of creosoted pine gates may be taken at one-half of that of iron gates, and from two-thirds to three-fourths of that of oak gates.* He also gives it as his experience that, for locks ranging from 45 to 65 feet in width, iron gates, with double plating, cost an average of 20 per cent. in excess of oak gates, and, for locks of about 40 feet in width, gates with an iron frame and creosoted planking cost an average of 15 per cent. in excess of oak gates. These conclusions are based exclusively on statistics obtained from the more important maritime canals of the Netherlands.

4. **Cost of Maintenance.**—Reliable and extensive data for general application on this point are not forthcoming. The writer's experience is that, in regard to greenheart gates, the cost of maintenance is practically nil. Gates of oak and pine are stated by Messrs. Brandt and Hotopp to require an annual upkeep expenditure of ½ to 1 per cent. of their prime cost. Some iron and steel gates are recorded as costing as much as 1 to 1½ per cent. Mr. Nelemans places the several materials in the following order as regards maintenance, commencing with the costliest :—Creosoted pine, iron, oak. He states, in this connection, that "the maintenance expenses of wooden lock gates exceed those of iron gates by 50 per cent.,

* Nelemans on "Iron and Wooden Lock Gates," *Min. Proc. Ninth Int. Nav. Cong.*, Düsseldorf, 1902.

and exceed those of gates with iron framework and planking by 25 per cent."

Owing to the variability of local practice, there is no absolute standard of comparison.

5. **Durability.**—As regards this point, the advantage, on the whole, lies with wooden gates. Salt water, especially if in any way contaminated with sewage, is extremely deleterious to ironwork. As has already been pointed out in Chapter iv., the metal, if unprotected, is speedily reduced to a condition resembling graphite or plumbago in structure. Painting, the preservative agency most usually adopted, is merely a temporary expedient calling for constant renewal, while the more expensive process of galvanising adds but a few years to the natural life of a gate at the expense of some reduction in the strength of the material. The life of an iron gate, under normal circumstances, can scarcely be expected to exceed thirty years, and the following are actually recorded instances of the rate of decay:—A pair of iron gates at a lock on the Dedemsvaart Canal in Holland,* constructed in 1880, were removed for repairs in 1894, when it was found that the framework was covered with a layer of rust which had to be scraped away, while the sluice paddles and their grooves were completely worn out so as to need replacing. The galvanised sheeting was intact, but it was deemed advisable to coat it with black varnish. A pair of gates at Glückstadt Harbour, on the Elbe, built in 1874, were condemned in 1902. Dock gates at Bremerhaven, erected in 1852, were removed in 1900 as completely worn out, the plates being eaten away below low water to a depth of $\frac{1}{8}$ of an inch, and the rivet heads either badly decayed or entirely destroyed.† Naturally, the life of an iron gate depends very largely on the amount of care devoted to its maintenance, and, in order to keep such gates in proper condition, they should be scraped, cleaned, and painted annually, or at intervals not exceeding three years. The lock gates at Terneuzen and Ymuiden are thus treated.

Apart from the attacks of sea worms (and some ports are apparently exempt from these pests), wooden gates, more particularly those of oak and greenheart, are extremely durable and need no attention. Mr. Blandy ‡ mentions the case of the old Waterloo Dock gates at Liverpool, constructed of oak, which, when removed on account of alterations and taken to pieces, were found to be in a perfect state of preservation after forty years' exposure to tide, wind, and weather. § The 100-foot greenheart gates at the Canada Lock of the same port were in active use for a like period, 1856 to 1895, and, when removed on similar grounds and taken asunder, were found to be in an absolutely sound condition and as good as on the day

* *Min. Proc. Seventh Int. Nav. Cong.*, Brussels, 1898, p. 325.

† Brandt and Hotopp on "Iron, Steel, and Wooden Gates," *Min. Proc. Ninth Int. Nav. Cong.*, Düsseldorf, 1902.

‡ Blandy on "Dock Gates," *Min. Proc. Inst. C.E.*, vol. lix.

§ The gates lay on the beach for several years prior to being broken up.

when they were built. Owing to the deepening of the lock, new verticals had to be introduced, but the old horizontal ribs were replaced, and are now doing duty as effectively as the new timber, with every prospect of an indefinite existence. The greenheart storm-gates of the Sandon entrance, built about the year 1848, were taken to pieces in 1902 and found to be in excellent condition. The Bramley-Moore Dock gates, of English oak, built about 1835, were overhauled in 1902; below the water-line, the wood was in perfect preservation, but decay had occurred in some sapwood in the upper part of the gate, which had to be made good. The greenheart gates at the Delamere Dock, at the entrance to the River Weaver, were constructed in 1862. No repairs of any kind have been executed to them, and they are still in admirable condition.* The greenheart gates at the sea entrance to Hendon Dock, Sunderland, were constructed in 1866. With the exception of some caulking to the planking, no repairs have been carried out, and the gates are still practically as good as new.†

Where the timber is of less trustworthy character the same durability cannot be reasonably expected. Continental gates often contain a large proportion of ordinary pine and pitchpine, timbers which do not possess the lasting qualities of oak and greenheart. It is not surprising, therefore, to find that the average life of such gates is about twenty-five years, though, with constant care, Mr. Nelemans states that very good results have been obtained, after nearly forty years' trial, with creosoted pine, "although the gates concerned are not usually worked, excepting those in the old Ymuiden Locks. It should be observed, however, that gates which are nearly always in their recesses do not last longer than those which are regularly worked." ‡

The one really weak point in the argument for the longevity of wooden gates is their liability to the depredations of sea worms. The *Limnoria terebrans* and the *Teredo navalis* (*vide* Chapter iv., p. 151) are two extremely persistent and troublesome borers, but they do not infest sewage-polluted waters, at any rate to any serious extent, and greenheart appears to be little, if at all, susceptible to their ravages,§ possibly on account of a poisonous oil which it contains. A splinter of greatheart in the flesh will

* Hunter on "Dock Gates of Greenheart and Steel," *Int. Nav. Cong.*, Düsseldorf, 1902.

† *Ibid.*

‡ Nelemans on "Iron and Wooden Lock Gates," *Int. Nav. Cong.*, Düsseldorf, 1902.

§ The testimony on this point is not altogether unanimous. Mr. Squire states that "Greenheart offers, perhaps, the best resistance to the ravages of the *Pholas* and *Limnoria* on the exterior, and of the *Teredo* on the interior, of the wood, but it is by no means invulnerable. In the Bombay Docks, greenheart gates were freely attacked by all these animals, especially on the seaward side of the gates and on the underside of the ribs. For the first few years they appeared only in the corners of the large ribs where the less mature timber would be found, but ultimately they penetrated the heart-wood."—On "Lock Gates," *Ninth Int. Nav. Cong.*, Düsseldorf, 1902.

certainly produce a nasty, festering wound, difficult to heal. There are sundry precautions which may be adopted to minimise the mischief caused to gates by marine vermin. They have already been dealt with in the chapter on Materials of Construction.

6. **Strength.**—Another respect in which timber gates have an advantage over iron gates is their more solid construction and consequent greater ability to stand the peculiarly rough usage to which dock gates are unavoidably subjected. Entrances sometimes have to be closed in the face of a strong outflow of water, and at such times there is a tendency for the gates to strike the sill with considerable force, in spite of the restraint of check chains and springs. Occasionally, moreover, the leaves do not reach the sill simultaneously, and the top part of the leaf, meeting with no support, is jerked violently forward. An instance is on record where, in the absence of a check chain, the topmost outer corner of a gate at Birkenhead was projected momentarily some 10 or 12 feet out of plumb.* The leaf then recoiled, and, fortunately, mitred fairly with its neighbour without further mishap; but the shock must have been tremendous, and nothing save the elasticity and flexibility of a wooden frame, with broad tenoned joints, could possibly have withstood the strain. As another instance of the almost disastrous nature of some of the conditions to which a gate may be subjected, mention may profitably be made of a serious accident which quite recently befell a pair of wooden gates at Liverpool, closing a passage 90 feet wide between two adjoining docks. One of these is a half-tide dock, in which the water is allowed to fall with the tide for some hours after high water. The passage gates were carefully mitred at the turn of the tide, and attention was directed to them until a steadily increasing head of some 15 or 18 inches of water was registered. At this point, being night-time, they were left, apparently secure. Unfortunately, by some carelessness or oversight, water for levelling purposes was run off from the inner dock at too rapid a rate, and the accumulated head was dissipated, with the result that the gates parted. Shortly afterwards, when the sluices were closed and at a time when the tide was ebbing fast, the gates came together again, probably with some impact, certainly imperfectly, and in such a way as to cause nipping between the outer edge of one mitre-post and the inner edge of the other. The falling tide soon produced a fresh head of 4 feet or so, at which point the foully mitred gates yielded with a loud crack. The alarm being raised, immediate steps were taken to avert any further evil consequences. The gates were found to be badly strained, and one leaf had to be taken into the graving dock for repairs. Despite the resistance of the connecting straps, the topmost ribs were torn out of the heel-post, and the upper portion of the latter was so split as to need splicing with new timber. The nipped edge of the mitre-posts were also badly detruded. However, the damage was soon made good at a moderate cost, and though the incident, at first sight, demonstrates

* This is the authentic statement of an expert eyewitness.

the vulnerability of timber gates, yet it may be claimed that the injury was far from vital, that the repairs were speedily effected, and that in undergoing a similar experience, the damage to a pair of iron gates would have been well-nigh irreparable. The veriest trifle, indeed, may cause them serious if not fatal injury, owing to the thinness of their skins, the rigidity of their rivetted joints, and the delicate adjustment of their buoyancy chambers. Several instances might be cited, but the following extract,* relating to a pair of iron gates at Limerick, will suffice:—

"About 1867, the bottom plates were unaccountably injured. The air-cells filled with water, which it was found impossible to eject, as no provision had been left for pumping. The result was a total loss of buoyancy, the whole weight of the gates being thrown on the bottom pintles and rollers. Temporary repairs to the damaged plates were effected by divers, and sluice doors were placed over the inlets on the river face, so that the effect contemplated by the designer was reversed, the air-cells and water-cells changing their functions. This arrangement was partially successful, but had the disadvantage of imparting such an excess of buoyancy to the gates that during rough weather, at spring tides, they were nearly floated off the hinges, whilst at neaps as many as twelve men were often required to move them. The state of things grew worse, for the roller carriages became disabled under the undue stress, causing the gates frequently to jamb in the closing, allowing the water to leave the dock." After this, it is not surprising to learn that the estimated cost of repairs rendered an entirely new pair of gates advisable.

After receiving a number of reports on the relative merits of wood and iron gates, followed by a general discussion, the Ninth International Navigation Congress, sitting at Düsseldorf in 1902, came to the conclusion that no definite opinion could be expressed as to the preference to be accorded to wood or iron gates, the question depending almost entirely upon local considerations. They adopted a further conclusion, however, "that for locks of great width, iron gates offer the advantage over wooden gates that they can be more easily constructed with suitable stiffness and durability, more readily and expeditiously moved, and more expeditiously and less expensively installed and removed." The reader will be able to form his own conclusions from the evidence which has been laid before him.

From a German official of public works, Herr Fülscher, comes a novel suggestion for *compound gates*. Since wood is durable under water and perishable above, while for iron and steel the conditions are reversed, Herr Fülscher advocates the employment of each material in the situation which is particularly favourable to it, so that the lower part of a gate would be of wood and the upper part of iron. The idea is ingenious and plausible, but no attempt has yet been made to carry it into effect, and there are several serious difficulties in the way of its realisation. It would manifestly be an

* *Min. Proc. Inst. C.E.*, vol. xcvii., p. 336.

unsuitable design for localities in which there was any important change in tidal level, and it is chiefly in such places that gates are required.

Classification of Gates.—Gates may be most efficiently classified as—

(α) Those consisting of a single leaf.

(β) Those having double leaves.

In the former case the axis of rotation may be either horizontal or vertical; in the latter, it is necessarily vertical.

Single Leaf Gates.—A single leaf gate with a vertical axis can only be advantageously employed for a very narrow waterway. When swung back to allow a passage for vessels, it occupies a side recess of considerable extent, rendering the entrance or lock unduly long and correspondingly expensive. Such a gate is rarely, if ever, constructed for dock work, and is almost entirely confined to canals. The following conclusion, voted by the International Navigation Congress sitting at Brussels in 1898, sums up the advantages and disadvantages in a clear and concise manner.

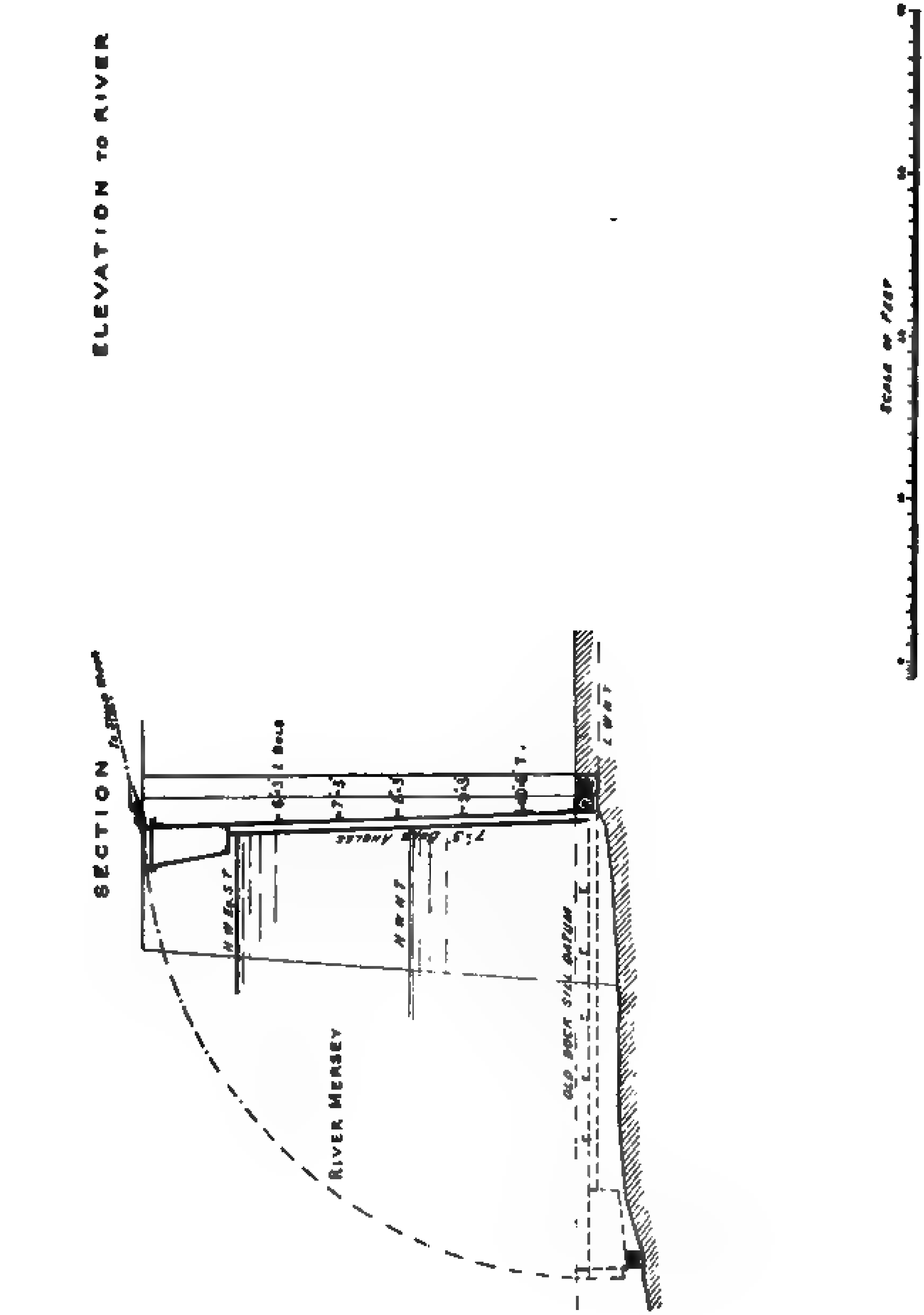
"Single gates, turning on a pivot, claim the attention of engineers. Notwithstanding the lengthening of the lock which they involve, they are not more expensive than mitred gates; they are subject to less strain, cause less loss of water, and are more easily adjusted, repaired, and replaced; and their working is simpler and more regular. Nevertheless, the great expenditure of water, and the increase in the period of locking, resulting from the elongation of the chamber, are inconveniences which, as regards the lower gates, counterbalance and even outweigh the advantages mentioned above." *

A single leaf gate, however, with a horizontal axis, is capable of much wider application. It turns upon a hinge or pivot, fixed slightly below the level of the sill of the entrance. When open, it lies prone upon a platform, below and outside the sill, so contrived that no part of the gate in this position projects above the sill level. The process of closing consists in raising the outer edge of the gate until it is vertically over the pivot. When this is done, the gate has a bearing against the two side quoins and against the face of the sill. The raising may be effected by means of a suitable attachment of wire ropes or chains, leading from the topmost member to a winch or other winding apparatus on the quay, but the action can be aided to a considerable extent by the formation of watertight compartments within the gate, the flotation power of which reduces the external lifting force required.

Messrs. Clover, Clayton & Co., of Birkenhead, have a gate constructed on this principle at one of their private graving docks. It is illustrated in figs. 247 to 249.† It closes an entrance of rather more than 40 feet mean width and its height is 27 feet 7 inches. The framing consists, on the inside, of four horizontal tiers of bulb-angle iron, ranging from 6 by 3 inches to 9 by 3 inches, with a lowermost tier of 10 by 6 inches bulb tee iron; and

* *Proceedings*, p. 638.

† *Vide* Brodie on "Dock Gates," *Min. Proc. L.E.S.*, vol. xviii.



Figs. 247, 248, and 249.—Single Leaf Gate at Birkenhead.

on the outside, of 20 vertical bulb-angle irons, each 7 by 3 inches, spaced about 2 feet apart. The plating between the framings varies in thickness from $\frac{1}{4}$ inch at the top to $\frac{3}{8}$ inch at the bottom. The topmost member is arranged as an air chamber, and it also serves the purpose of a stiffening girder. The meeting surfaces of the gate, the sill and the jambs, consist of pieces of pitchpine, faced with strips of indiarubber, 2 inches wide and $\frac{3}{8}$ inch thick, to secure watertightness. The method is apparently very effective, and the joint a perfectly durable one, as the author found from personal inspection. At the end of seven years the indiarubber, which is fastened by copper nails, was quite undeteriorated. The gate is swung on two hinges, having pins 4 inches diameter. A sluice at each side of the lowermost panel completes the equipment of the gate.

The success attending this type of gate, of which the foregoing is probably the principal existing example in this country,* is sufficient to warrant its introduction on a larger scale. The main objections attending such a step are the necessity for a platform deep enough to contain the buoyancy chambers, and the possibility of some unseen obstacle preventing the gate from falling back to its full extent, and thereby endangering vessels passing over it. These disadvantages cannot be considered insuperable. Special recesses might be formed in a comparatively shallow platform to receive the buoyancy chambers, and these would be kept clear of deposit by an efficient system of sluicing. An additional element of strength could be imparted to the gate by the adoption of a sill curved in plan, to which the turning axis would be tangential at its centre, as exemplified in the lower portion of a railway carriage door. This would entail somewhat longer hinges at the sides, in order to cover which, and the curved profile of the gate, the sill would also require to be curved in elevation—an objectionable arrangement for passages frequented by flat-bottomed vessels.

Gates with two Leaves.—By far the more general method is that of gates in two symmetrical leaves, each a little longer than the semi-width of the waterway, meeting, when closed, at its centre line in such a way as to afford one another mutual support by pointing in the direction of the impounded water.

Of this class of gate there are two varieties, representing distinct forms of construction, viz.:—

(α) Those with horizontal girders.
(β) Those with vertical girders.

The first case represents the type most commonly met with in British ports. It is founded on the principle of the arch, and consists essentially of a series of horizontal ribs or girders. In timber structures, these are grouped more or less into "cesses" throughout the height of the gate, the

* The author is only aware of one other example, viz. :—A gate closing the entrance (35 feet wide) to a graving dock at Port Dinorwic, North Wales.

intermediate spaces being faced with planking. In the larger gates, the cesses are not continuous from one end of the leaf to the other, but are intersected by verticals which divide each leaf into a series of *voussoirs.* In iron gates the horizontal members are single girders, continuous throughout, with intermediate connecting pieces, or stiffeners, and plating. There are at least two continuous vertical members in both kinds of gate—the *heel-post,* or axis of rotation, set in the hollow quoin of the entrance, and the *mitre-post,* forming the abutment at the outer end of the leaf. In timber gates the horizontal ribs are tenoned into and between these two main verticals, and for small gates they are sufficient. But for medium sized leaves of arched form, ranging from 30 to 40 feet in length, an additional vertical called the *middle-head* is economically introduced, dividing the leaf into two voussoirs. For larger entrances still, the middle-head can be duplicated, the two posts being distinguished as the *heel-middle-head* and the *mitre-middle-head,* according to their respective positions. In extreme cases, where the length of the leaf reaches from 50 to 60 feet, three intermediates will be required.

In the second type the method of construction is reversed, and the principle of the beam adopted. There are only two continuous horizontal members, one at the sill level, forming a watertight abutment, and the other at the summit of the gate. Between these are set a series of verticals at regular intervals from heel-post to mitre-post. The intervening space is made good with planking or plating, as the case may be, the thrust upon which is transmitted by the verticals to the upper and lower transoms, and these, accordingly, receive the whole hydrostatic pressure in a ratio to be determined later. For curved timber gates the verticals may, in certain cases, be arranged in contiguity as the voussoirs of an arch, but the necessity for having them in such close contact is remote, and the system is more generally characteristic of flat gates, such as are in evidence at Dunkirk, on the North Sea Canal and elsewhere.

For the sake of offering some basis of comparison of the merits of the two systems, it may be remarked that the vertical type is more readily adaptable to the accommodation of large sluice openings in the gate itself, as these can be arranged between the verticals without impairing the strength of the framing. On the other hand, the horizontal system has obvious advantages in respect to the more effective distribution of the material, and, in the case of wooden gates, at any rate, it undoubtedly represents the soundest and most economical form of construction.

Storm Gates.—A class of gates differing in function, rather than in mode or form of construction, is that known as storm or flood gates. They are employed in entrances subjected periodically to floods or to extraordinarily high tides accompanied by cyclones and tempestuous weather. During such periods it is often necessary to exclude part of the tidal water from a dock, and the gates consequently point in the opposite direction to those used for impounding water. From the nature of their duties it is evident

that they call for exceptional strength and careful construction. In some instances a ship caisson is employed for the purpose, especially when the circumstances are of rare occurrence.

Strut Gates are auxiliary frames or shores which support the main ebb gates in their closed position and enable them to withstand a slight head on the outer face, and to resist the onset of waves at or about high-water level. They assist ebb gates to act to a certain extent as storm gates, and are accordingly found to be a useful adjunct in exposed situations (fig. 201).

Stresses in Gates.—Proceeding now to an investigation of the stresses to which gates are subjected, it will be found on consideration that the causes to which they are due may be ranged under five heads:—

1. The excess of water pressure on the inner side, or back of the gates, when closed.
2. The tension of chains, or the thrust of rams, during the operations of opening and closing.
3. Concussions and irregularities of movement in consequence of unseen obstacles and incomplete control of the motive power. In this connection it is to be noted that a strong current sometimes forms a very great part of the motive power.
4. Wind pressure and the impact of waves.
5. Collisions with passing vessels.

Of these, the three last are of a more or less abnormal nature, and their magnitude cannot be estimated with any degree of exactitude or certainty. Nevertheless, they constitute very potent factors in the determination of the life and stability of a gate. In boisterous weather, not only do external waves break against the outer face of a gate in a succession of shocks of varying intensity (the effect at high water being especially destructive), but even the water confined within the dock will often become considerably agitated, especially if there be any extensive area exposed to the action of the wind. This last named agent also exerts direct unbalanced pressure upon the surface of the gate above the water line, but as the unimmersed portion is, as far as possible, constructed in openwork, the result is minimised.

During the operations of opening and closing, the gates are liable to jars and shocks from contact with sunken obstacles, from abrupt stoppages due to occasional fluctuations in hydraulic pressure, where such is employed, from the sudden impetus of wind, wave, and tidal current, and even from irregularities in, and silt accumulations upon, roller paths. If the tide be running out with any degree of swiftness, a rapid current is generated in narrow entrances, in which it is difficult with rams, and almost impossible with chains, to prevent the gates from striking the still with some force, instances of which have already been noticed.

Collisions are occurrences more or less frequent during the time the entrance or passage is being worked. Accordingly, it is very essential that the open gate should be completely recessed beyond the face line of the

waterway. Even when this point is carefully attended to, it is impossible to avoid chance contact, and the abrading action can only be neutralised by the provision of stout and ample fendering. Perhaps the best form of gate to suit these conditions is that with a straight inner face, and when the gate is segmental, it is desirable that the fendering should be arranged so as to form a chord to the segment. This gives a straight lead to shipping, and prevents the arch voussoirs from receiving a pressure from the quarter in which they are least fitted to resist it.

For all these and other varieties of stress, more or less intermittent in character, uncertain in direction and unknown in amount, provision can only be made in a crude and wholesale manner by the employment of a high factor of safety. It is not too much to say that the actual strength of a gate should be at least ten times, and, in certain cases, as much as twenty times, in excess of its calculated requirements under normal statical conditions. This factor of safety attains its higher values in the case of wooden gates, where the material has a wide range of strength. The resistance of iron and steel can be estimated with greater exactitude, and therefore admits of a closer approximation.

Statical Forces.—The only statical forces are those called into action by the excess of water pressure on the back of the closed gate, and by its own weight, and it is inevitable to limit the calculations for the stability of gates to a consideration of these simple elements. Calculations are sometimes carried out to a theoretical nicety, which, however ingenious and interesting, is of questionable expediency in view of the wide margin of safety ultimately adopted. Inordinate detail in calculation entails two evils; it not only involves a waste of time, but leads to an exaggerated view of the accuracy and importance of the result. In investigating, therefore, the internal stresses, caused by external agency, an attempt must be made to steer a middle course between the Scylla of useless refinement and the Charybdis of superfluous strength.

When a pair of dock gates is closed and the water within the dock is at a higher level than that outside, the horizontal external forces at work are four in number:—

1. The resultant pressure of the water against the back of the gate.
2. The mutual reaction of the mitre-posts.
3. The reaction of the hollow quoins on the heel-posts.
4. The reaction of the sill against the bottom of the gate.

The conditions, in fact, are those of a loaded vault closed at one end.

It will, perhaps, be preferable to consider primarily the joint effect of the first three forces, and then to estimate the modification caused by the fourth force, which does not in any way affect the relationship existing between the other three. The forces being symmetrical for each half of the gate, it will only be necessary to deal with a single leaf in each case.

1. *Resultant Water Pressure.*—This force is completely defined, since it is known in magnitude, line of action and sense.

(α) *Magnitude.*—The weight of a cubic foot of fresh (distilled) water is 1,000 ozs. av. or $62\frac{1}{2}$ lbs. The weight of a cubic foot of salt water depends upon its impurities, which vary in different localities. Within the limits of the British Isles, it ranges from 1,000 to 1,025 ozs., and will be taken here at its maximum value of 64 lbs., and symbolised, when necessary, by the letter w. The intensity of water pressure on the back of a gate increases from zero at the surface level to an amount at any other level due to the height of the column of water above it. Thus, at a depth of 10 feet below the surface, the pressure is $10\,w = 640$ lbs. per square foot. When there is no water on the front of the gate this represents the resultant intensity of pressure at that level. If the height of the water behind the gate be designated h, then the mean intensity of pressure is $\frac{w\,h}{2}$, and the total resultant pressure per lineal foot of gate,

$$P = \frac{w\,h^2}{2}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (45)$$

If there be a height, h_1, of water at the front of the gate, it is manifest that this expression must be modified into

$$P_1 = \frac{w\,h^2}{2} - \frac{w\,h_1^2}{2} = \frac{w}{2}(h^2 - h_1^2). \quad . \quad . \quad . \quad (46)$$

(β) *Line of Action.*—The centre of pressure is situated at the level of the centre of gravity, D, of the triangle, A B C, representing the distribution of pressure (fig. 250). That is to say, the line of action is horizontal, at one-third of the height of the water. When there is water on only one side of the gate, this applies to the resultant pressure also. With water on both sides, the resultant pressure will act at a point determined by the centre of gravity of the quadrilateral, A C F D (fig. 251), formed by deducting the lesser triangle of pressure from the greater. If the gate be curved, the line of action will be normal to the curve.

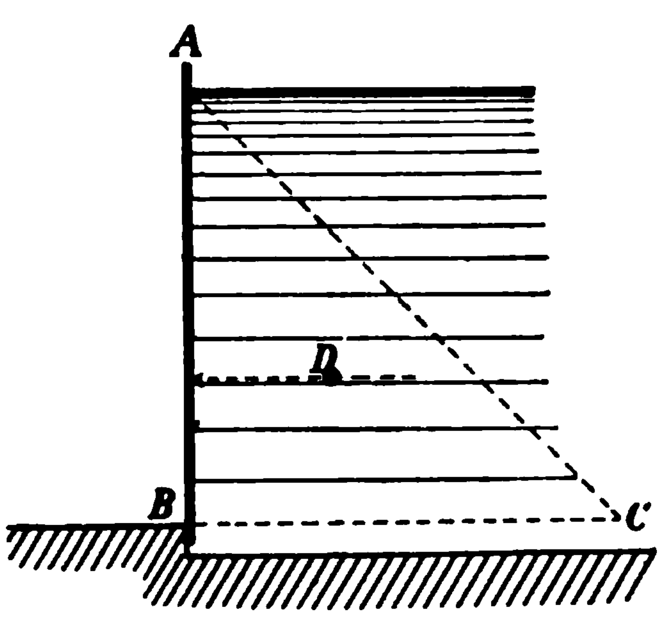


Fig. 250.

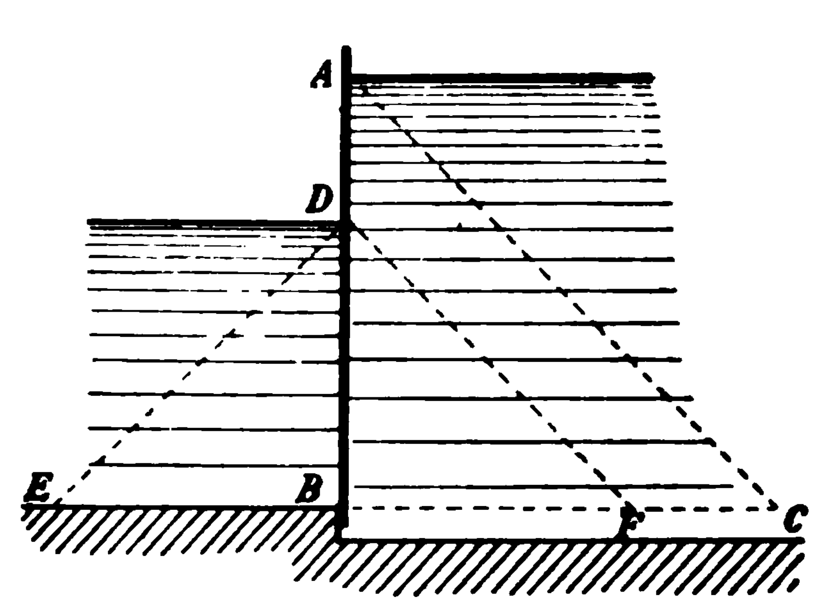


Fig. 251.

(γ) The *Sense* is obvious, being always towards the gate.

2. The *Reaction of the Mitre-posts* is a force perpendicular to the plane of their abutting surfaces, and, therefore, also perpendicular to the longitudinal axis of the waterway. Even with perfectly fitting gates, it would not

be justifiable to assume that the line of action of the force passed through the centre of the meeting faces, and, in practice, it must inevitably happen that the gates are the veriest trifle too long or too short, in either of which cases the gates will *nip* one another; if too long, on the inner edge, and if too short, on the outer edge, of the mitre-post (fig. 255). Nipping may also be due to the accidental intrusion of some small floating substance, such as a chip of wood. Under these circumstances, the line of action would pass near to the inner or outer edge of the mitre-post. For the present, however, the assumption will be made that it bisects the meeting surfaces.

3. If friction be left out of account, the *Reaction of the Hollow Quoins* will pass through the centre of the heel-post, and further (the three forces being in equilibrium), through the point of intersection of the other two forces, and these two points are sufficient to determine its line of action. When the gates, however, are just closed, and during the period in which the parts are taking up their respective stresses, there is some inevitable, albeit almost infinitesimal, yielding of the wooden heel-post, and a corresponding movement along the face of the rigid masonry, which brings into play a frictional force, R tan ϕ, where R is the thrust on the heel-post and ϕ the angle of repose of wood on stone. If r be the radius of the heel-post, the reaction of the hollow quoin will accordingly pass at a distance, $r \sin \phi$, from its centre. The deviation is generally slight, and, unless the thrust be very great, its effect may be ignored.

4. The *Reaction of the Sill* upon the lowermost horizontal member of the gate is frequently overlooked, but that it is capable of affording no inconsiderable assistance to a gate under pressure is manifest from the fact that it is quite theoretically possible to construct a gate deriving its entire support from the sill alone. This will be apparent from a glance at fig. 252, in which the top of the sill coincides with the centre of gravity of the water pressure against the gate. The latter, accordingly, is in critical equilibrium, which the least increase in its depth below the sill renders stable. The inconvenience, however—if not the impracticability—of providing so deep a sill, with a perfectly watertight joint, constitutes an insuperable objection to such an arrangement. The reaction of an ordinary shallow sill is not altogether easy to determine, but it may be considered in two ways. It may be deemed to raise the level of the centre of pressure, though, in this respect, its effect is scarcely appreciable. It may also be taken as exerting a moment about the top edge of the sill, contrary to and partially counteracting the moment due to the water pressure above the sill. This latter, however, would only be a legitimate aspect of the problem, provided

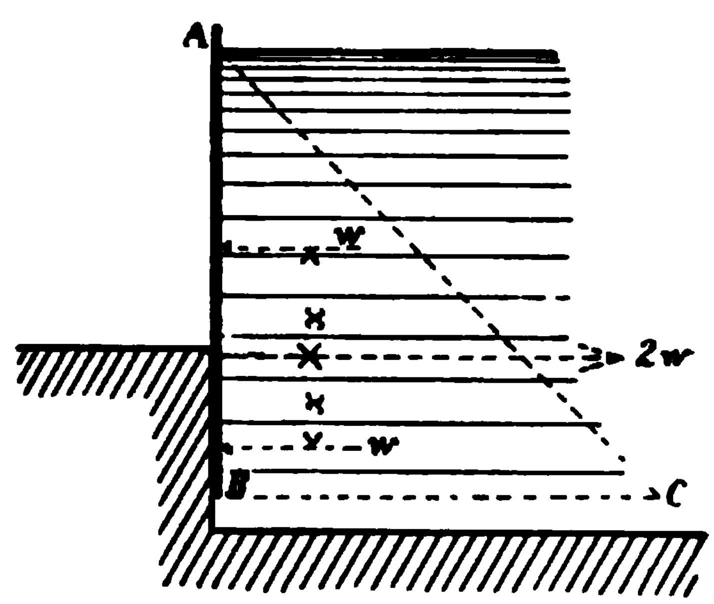


Fig. 252.

the gate were constructed mainly or altogether of verticals, for the joints between successive tiers of horizontal ribs are not, and cannot be supposed, capable of resisting transverse stress. In the absence of such joints, it is justifiable to state that the pressure on the watertight surface of the sill is sufficient to counterbalance, at least, the pressure on an equal height of the unsupported portion of the gate immediately above the sill. The case is that of a cantilever, the moiety of whose length is unsupported and loaded with a weight something less than the weight on the supported half. In flat gates of the vertical type, the sill plays a most important part, supporting, as will be seen later, two-thirds of the total water pressure upon the gate.

So much for the horizontal forces. The vertical forces are two in number:—

1. The dead weight of the gate, acting downwards through its centre of gravity.

2. The upward reaction, due either separately or jointly to (*a*) flotation of buoyancy chambers or the water pressure on the underside of the gate, (*b*) truckwheels or rollers bearing upon a platform, and (*c*) inclined straps connected with the top of the heel-post.

We need not consider these at greater length. Obviously, equilibrium can be secured by a suitable adjustment of the opposing forces. We proceed to deal with the more complex problem presented by the horizontal forces.

To find the Resultant Pressure on any Section of a Gate.

Fig. 253 represents the plan of one leaf of a pair of gates. P is the total water pressure upon the back of the leaf, assumed concentrated at its centre.

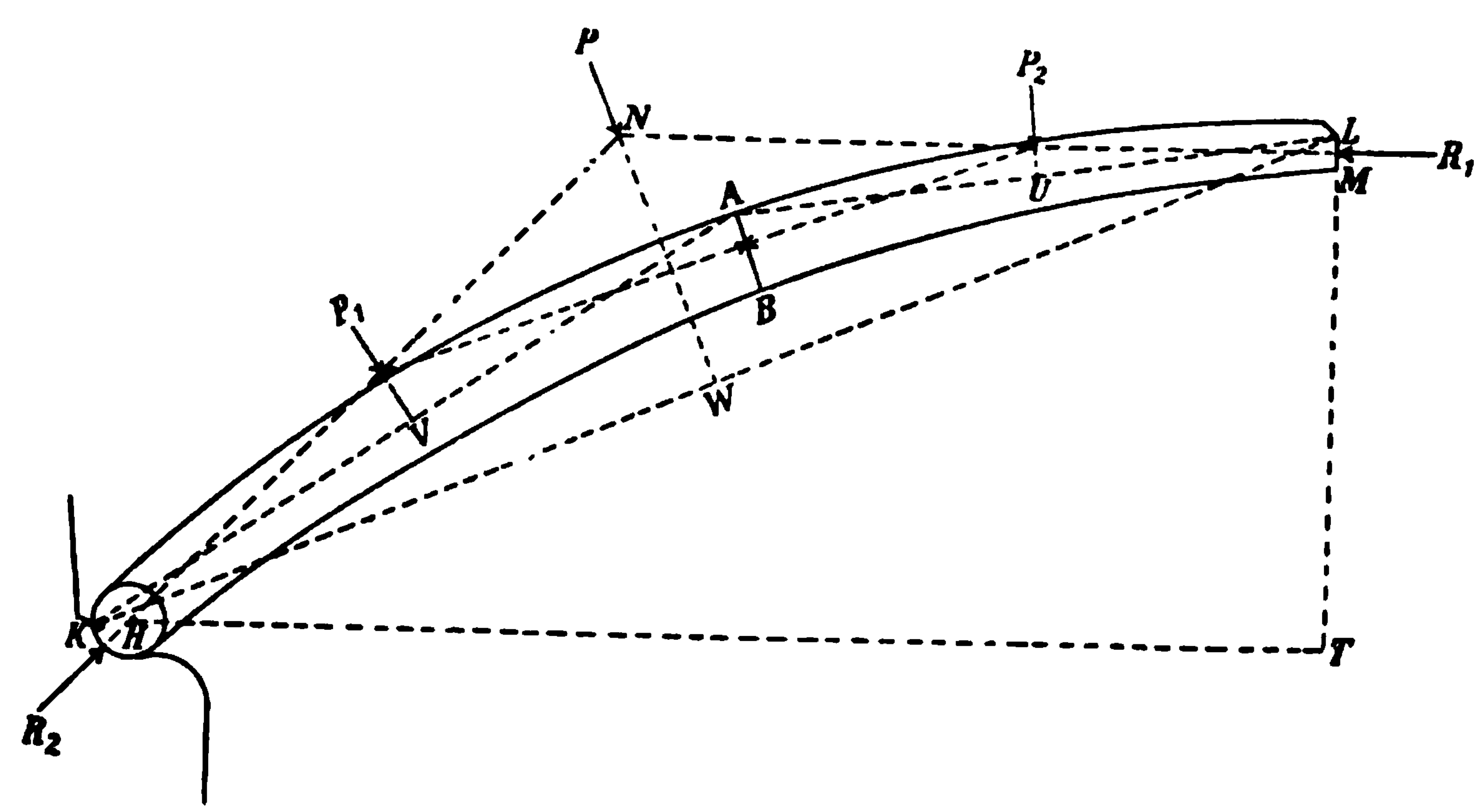


Fig. 253.

R_1 is the mitre reaction of the adjoining leaf, taken as passing through the centre line of the abutting surfaces. R_2 is the reaction of the hollow quoin

assumed to act through the axis of the heel-post. These three forces being in equilibrium, the triangle of forces, *o r s* (fig. 254), can be drawn, having its sides parallel to the forces, P, R_1, and R_2 respectively, and, since the magnitude of P is known, the magnitudes of the other two may readily be determined.

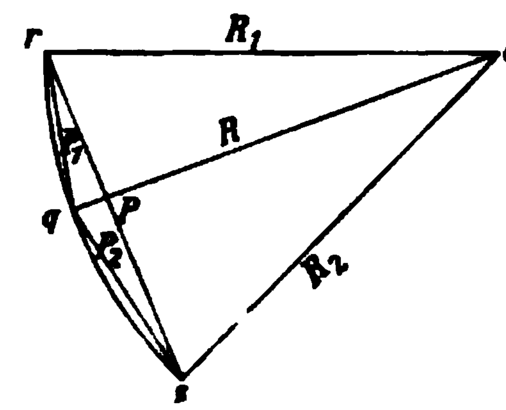


Fig. 254.

If, now, it be required to find the position and amount of the resultant stress across any section, A B (fig. 253), of the gate, we proceed as follows:—Join the point, A, to each of the two extremities, K, L, of the water-bearing surface of the leaf; bisect these lines at U and V respectively, and draw perpendiculars to represent the total water pressure on each section. Each section is in equilibrium under the action of these forces: in one case, the water pressure, the heel-post reaction, and the stress across A B; in the second case, the water pressure, the mitre-post reaction, and also the stress across A B, acting, of course, in the opposite direction. Since this stress must have the same line of action in both cases and must pass through the points of intersection of each of the other pairs of forces in order to fulfil the conditions of equilibrium, we have obviously only to join the two points of intersection to obtain the line of action of the resultant pressure at the section A B. Its magnitude may be determined by drawing a parallel line in the force diagram from the point, *o*, and completing the triangle by drawing the line representing the water pressure on either surface of the gate. Thus, in fig. 254, the mitre-post reaction being already determined, *r q* is the water pressure on the surface of the gate between the point, A, and the mitre-post, and *q o* is the stress across the section A B. Similarly, for the heel portion, a line, *q s*, might have been drawn parallel to the water pressure on that section. Thus the point, *q*, is not only obtained, but confirmed.

By taking a series of sections in this way, it will be found that the locus of the point *q* is sensibly the arc of a circle, and therefore that, except perhaps in the case of very flat gates, the resultant pressure is so nearly constant as to be justifiably considered so without serious error. Also, it will be found that the line of pressure is a circular curve. This is perfectly true for all gates which present the form of a continuous arc when closed. It is also approximately and practically true for all segmental gates varying between the straight line and the continuous arc, provided the versed sine or *rise* of the gates (T M, fig. 253) do not exceed one-fifth of the span. For a greater ratio of rise to the span the divergency of the line of pressure from the circular arc begins to be appreciable, and ultimately, in the case of the flatter gates, becomes very marked, so that it is necessary to find by trial a series of points through which the curve may be drawn. Fig. 255 shows the curves of pressure in a segmental gate for a central reaction at the mitre-post and also in case of nipping on the inner or outer edges of the mitre-post.

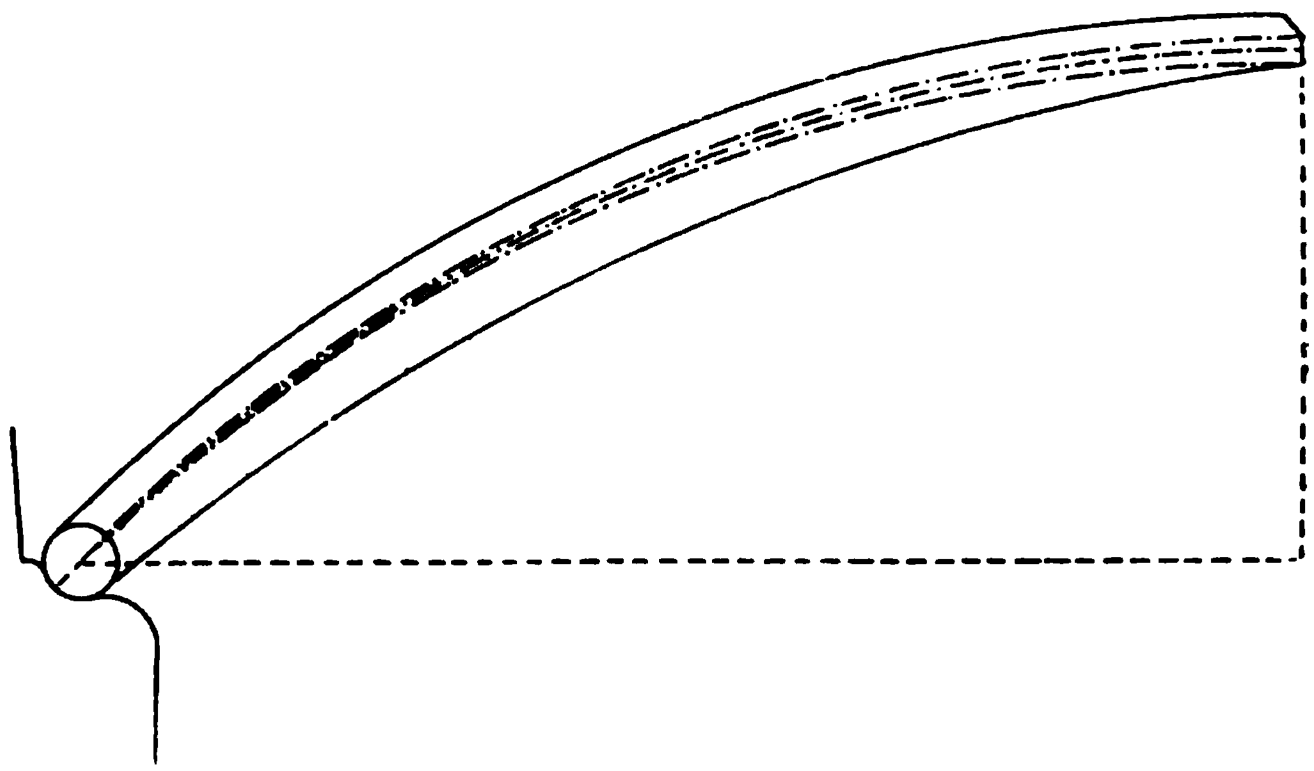


Fig. 255.—Range of Position of Line of Pressure due to Nipping.

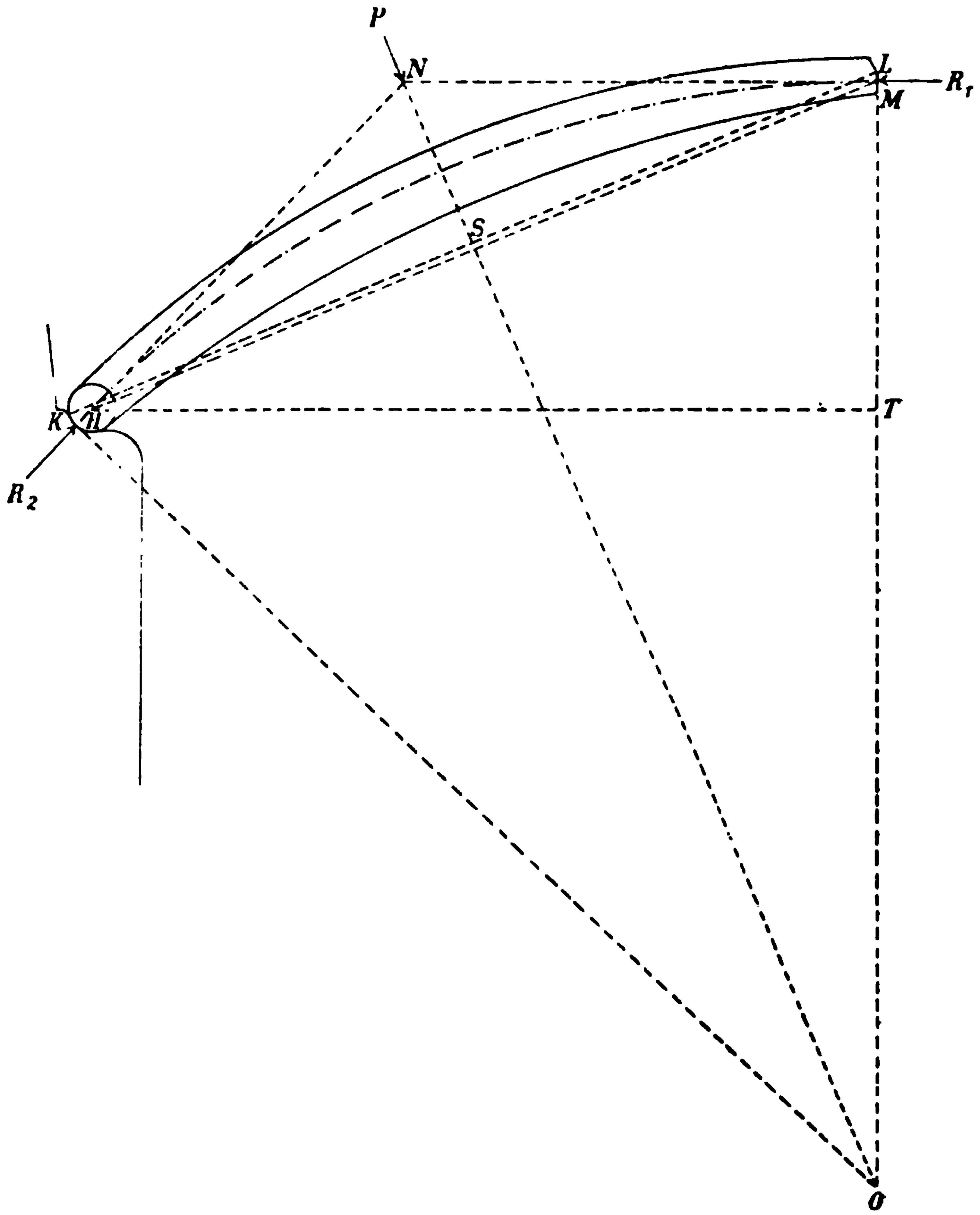


Fig. 256.

Another method of procedure, which has the advantage of including both diagrams in a single drawing, is as follows:—From one extremity, K (fig. 256), of the water-bearing surface of the leaf draw, perpendicular to the direction of the heel-post reaction, a line, K O, to intersect at O, the centre line of the passage, which itself is perpendicular to the mitre-post reaction. In this way a triangle, K L O, is formed, having its sides respectively perpendicular to the lines of action of the forces and therefore proportional to their magnitudes. And as P, the total water pressure, is measured by the length of the leaf, K L, multiplied by $\frac{wh^2}{2}$, so the heel and mitre-post reactions are K O $\times \frac{wh^2}{2}$ and L O $\times \frac{wh^2}{2}$ respectively and indifferently, for they are equal, as we have already seen. The reaction at any section of the gate can be obtained by drawing a line from O to that point of the water-bearing surface which lies on the section line in question. The length of this line multiplied by $\frac{wh^2}{2}$ gives the required reaction.

It may be convenient to obtain an expression for the value of the heel-post, the mitre-post, and the sectional reactions generally, in terms of the span and rise of the gates. This can be done with very close approximation as follows:—By the span of the gates is to be understood the distance between the centres of the two heel-posts and by the rise, the vertical distance from this line to the centre of the abutting surfaces of the mitre-posts. From an inspection of fig. 256 it will be seen that the line H M joining the centre of the heel-post to the centre of the mitre meeting surface—that is, joining the extremity of the span to the extremity of the rise—is practically and sensibly parallel to the line K L which connects the extremities of the water-bearing surface. Consequently we may imagine the angle H M T equal to the angle K L T without appreciable error. Then from similar triangles, H T M and O S L, we have

$$\frac{\text{O L}}{\text{S L}} = \frac{\text{H M}}{\text{M T}}.$$

Now, S L = one-half the total water pressure on the surface of the leaf $= \frac{wh^2 l}{4}$; and O L is to all intents and purposes a measure of the resultant pressure on any section—*i.e.*, R.

Again, H M is the length of the leaf minus the radius of the heel-post $= l - \rho$; and M T is the rise of the gate $= r$.

Substituting, we obtain

$$\text{R} = \frac{wh^2 l (l - \rho)}{4r}. \qquad (47)$$

If we choose to neglect ρ, which is a very small quantity in comparison with l, and to substitute for l^2 its component value $\frac{s^2}{4} + r^2$, we arrive at an

approximate expression for the resultant pressure in terms of the rise and span of the gates, viz. :—

$$R = wh^2\left(\frac{s^2}{16r} + \frac{r}{4}\right). \quad . \quad . \quad . \quad . \quad (48)$$

The following data apply to a pair of gates closing a 70-foot entrance:—

$l = 39{\cdot}75$; $s = 76{\cdot}3$; $r = 11{\cdot}16$; $\rho = 1$; $h = 30$—all in feet.

By formula (47)

$$R = \frac{64 \times 30 \times 30 \times 39{\cdot}75 \times 38{\cdot}75}{4 \times 11{\cdot}16}$$

$= 1{,}987{,}500$ lbs., or $887{\cdot}3$ tons.

By formula (48)

$$R = 64 \times 30 \times 30 \left(\frac{76{\cdot}3 \times 76{\cdot}3}{16 \times 11{\cdot}16} + \frac{11{\cdot}16}{4}\right)$$

$= 2{,}038{,}500$ lbs., or 910 tons.

The discrepancy between the two results, it will be observed, is less than $2\frac{1}{2}$ per cent.

Zones of Equal Pressure.—The surface of a gate may be divided into a series of zones, in which the total hydrostatic pressure is equal, in the following simple and graphical manner :—

With the height of the gate between the sill and the surface of the water as diameter, describe a semicircle (fig. 257). Subdivide the diameter into

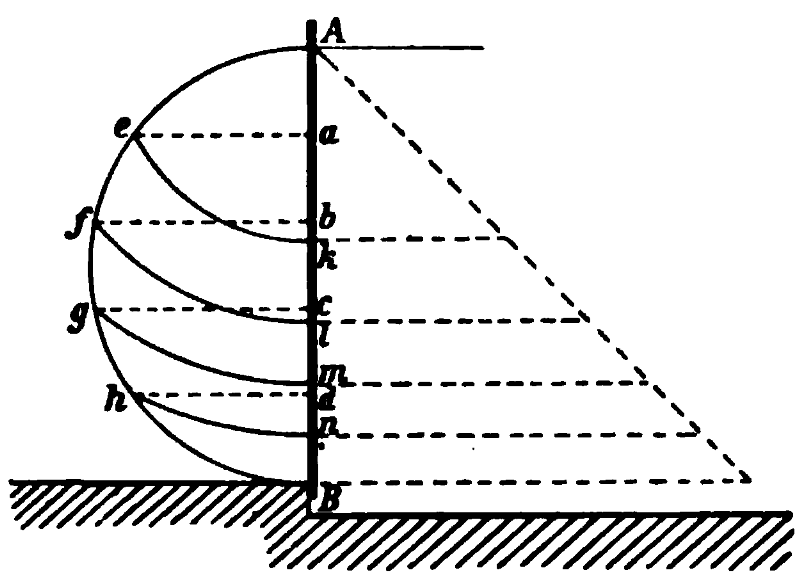


Fig. 257.

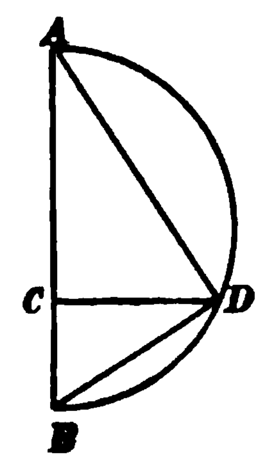


Fig. 258.

any number of equal parts (say five) by the points *a, b, c, d.* Through these points draw horizontal lines to the semicircle, intersecting it at the points *e, f, g, h.* Then, with A as centre, describe circular arcs *ek, fl, gm, hn,* cutting the gate surface at the points *k, l, m,* and *n.* A*k*, *kl*, *lm*, *mn*, and *n*B will then be a series of consecutive zones upon which the hydrostatic pressure is in each case equal to one-fifth of the total pressure upon the surface of the gate.

This may be proved by reference to fig. 258. There it will be seen that from similar triangles

$$\frac{AC}{AD} = \frac{AD}{AB},$$

$$\text{or } \frac{AC}{AB} = \frac{AD^2}{AB^2}.$$

That is (since the water pressure on any surface is proportional to the square of the depth), the pressure on A D is to that on A B in the same ratio as the depth A C to the depth A B.

Having divided the sectional area of material required into equal portions, the cesses or girders can be located at the centres of pressure of the respective zones.

Rise of Gates.—The ratio which the versed sine or rise of a pair of gates bears to the span varies in practice between the limits of one-third and one-sixth. The best proportion is a matter of individual experience and local requirements, rather than of theoretical calculation. Much depends upon the nature of the material of which the gate is constructed, its distribution and maximum resistance to stress, but the practical exigencies of the situation often outweigh them all in importance.

It has been stated that the most economical rises are about one-third and one-fifth for cylindrical and straight gates respectively.* But gates are rarely constructed with parallel faces, and the disposition of the material may be, and often is such, that the longitudinal axis, which is the true curve of the gate, follows a path in no way concentric with either of the faces. Further, it should be noted that mere economy in gate construction is a question of minor importance to those of stability, durability, and convenience. A great rise, combined with cylindrically-curved backs, calls for long and deep recesses in the side walls, and exposes a large gate surface to contact with passing vessels. On the other hand, at graving dock entrances the rise of the gates adds somewhat to the available length of the dock.

Considering the question for a moment merely from the point of view of the amount of stress due to different ratios of rise to span, let us refer to the closely approximate formula already devised for the value of the resultant stress in terms of the rise and span of the gates, viz. :—

$$R = w h^2 \left(\frac{s^2}{16 r} + \frac{r}{4}\right).$$

Re-arrange and let $r = v s$, so that v may have any value, integral or fractional, of which the latter alone calls for serious consideration. Then

$$R = \frac{w h^2}{2} \times \frac{1 + 4 v^2}{8 v} s.$$

In this equation we have an expression for the resultant in terms of the water pressure per unit length of the gate $\left(\frac{w h^2}{2}\right)$, multiplied by a coefficient involving the rise and span of the gates only.

Now assign to v a series of values ranging from ·1 to 1—that is, from $\frac{1}{10}$ to unity—and calculate therefrom the corresponding values for the coefficient

$$\frac{1 + 4 v^2}{8 v}$$

* *Min. Proc. Inst. C.E.*, vol. xviii., p. 474 ; vol. xxxi., p. 344.

The results form a series of co-ordinates from which the curve in fig. 259 has been plotted. The line A B constitutes the span, and along it have been marked off distances corresponding to the ratio of rise to span. From an inspection of the figure we see that the resultant pressure is least with a ratio of $\frac{1}{2}$, and that it increases in amount with any change from this ratio. The increment is comparatively small as the value of v approaches unity, but, as it approaches zero, the rate of increase is very rapid, becoming ultimately infinite. With a rise equal to $\frac{1}{3}$ span, the excess over the minimum is inconsiderable, and thence to a rise of $\frac{1}{5}$, it is but moderate, but for rises beyond $\frac{1}{6}$, the value of the coefficient becomes excessive.

In conjunction with the question of the total amount of stress, it must be borne in mind that the pressure between the gate framing has to be taken by metal plating and wood planking, as the case may be, and that there is a practical limit to their effective and useful resistance.

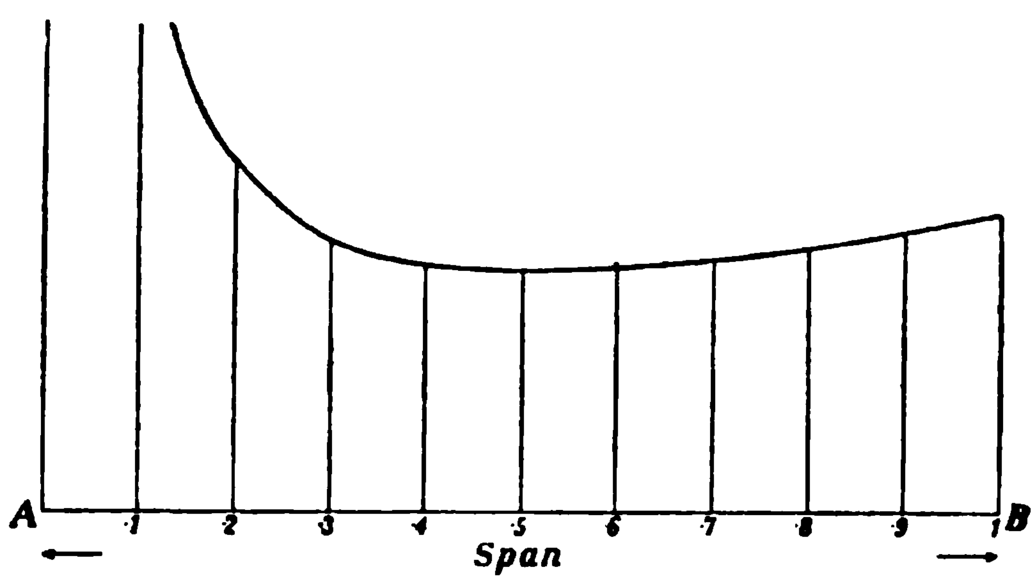


Fig. 259.

Taking everything into consideration—design, material, permissible stress, contingencies of manufacture—no definite rule can be laid down beyond the statement of the usual range already given.

Analysis of the Resultant Pressure.—Having obtained an expression for the resultant pressure on the cross-section of a gate, we now proceed to consider it with reference to its point of application.

The simplest, and theoretically ideal, gate would be that in which the line of pressure passed through the centre of gravity of successive cross-sections. In this way the joints would simply be called upon to sustain direct compression, uncomplicated by any bending moment. It has been pointed out that this does not necessarily imply that the back and front of the gate would be circular arcs concentric with the line of pressure. A straight gate might be constructed to fulfil the required condition by a suitable adjustment of the material, so that the centre of gravity of each section fell upon the line of pressure.

In most cases, however, practical considerations cause the axis of the gate to deviate more or less from the ideal curve.

Fig. 260 is the plan of a portion of a gate leaf, A A being the longitudinal axis—*i. e.*, the line passing through the centres of gravity of successive sections—and L M a line perpendicular thereto.

Now, let x be the point of application, in the curve of pressures, of the resultant, R, acting on the plane of section, L M. Then, if the angle between the line of action of R and the sectional line L M be designated θ, the force, R, may be resolved into two component forces—viz., R sin θ, parallel to the axis, A A, and R cos θ, at right angles to it. The former is a direct compressive stress and the latter a shear.

By introducing two opposite forces at the point O, each equal to R sin θ, a step which in no way interferes with the equilibrium of the section, we may conceive the line of action of R sin θ transferred to the axis, A A, provided that this change be taken in conjunction with a couple, the moment of which is R x sin θ, and which tends to turn in a clockwise direction.

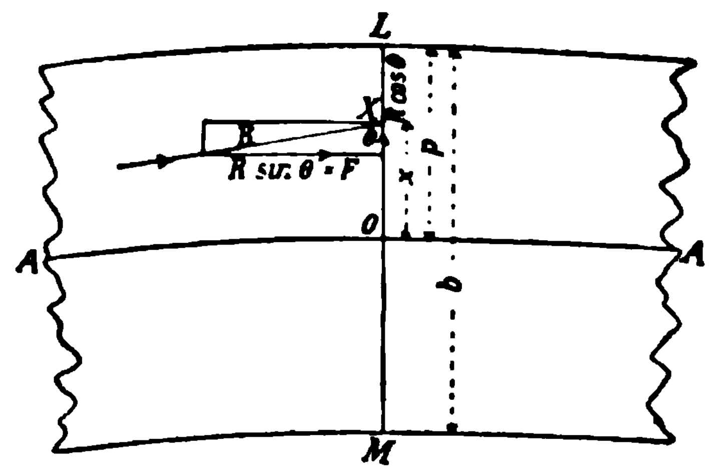


Fig. 260.

The section, L M, is thus subjected to the action of the following forces :—

1. A shearing stress, R cos θ, along L M.

2. A direct compressive stress of uniform intensity throughout the section, the total amount of which is R sin θ.

3. A bending moment, R x sin θ, producing compression from O to L and tension from O to M, both these stresses varying in intensity and increasing with the distance from the neutral axis at O.

The force R cos θ, being a simple shear, calls for no further comment. With regard to the force R sin θ, it will simplify the notation in the ensuing investigation if, from this stage, we symbolise it by the letter F. If the area of section be A, the uniform intensity of direct compression is $\frac{F}{A}$, and if the vertical depth be taken as unity, it is $\frac{F}{b}$, where b is the breadth of section = L M.

The intensity of stress at any point in the section due to the bending moment, F x, may be obtained from the well-known relationship,

$$\frac{f}{y} = \frac{M}{I},$$

where y is the distance from the neutral axis of the fibre sustaining the stress intensity, f, M is the moment of resistance, equal to the bending moment, F x, and I is the moment of inertia.

The greatest intensity of stress is manifestly that in the outermost fibres, at L, where the maximum compressive effect of the bending moment is added to the direct compression. Indicating the distance, O L, by the letter p, and taking the depth as unity, the intensity of stress due to direct compression is

$$f' = \frac{F}{b}, \quad . \quad . \quad . \quad . \quad . \quad (49)$$

and that due to bending is

$$f'' = \frac{F x p}{I}; \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (50)$$

whence the total intensity of stress at L—

$$f = f' + f'' = F\left(\frac{1}{b} + \frac{x p}{I}\right). \quad . \quad . \quad . \quad . \quad (51)$$

At the point M, on the other side of the neutral axis, we must give the tensile stress, f'', a negative value, and the equation becomes

$$f_0 = f'_0 - f''_0 = F\left(\frac{1}{b} - \frac{x(b-p)}{I}\right). \quad . \quad . \quad . \quad (52)$$

Graphic Representation of Internal Stress.—Fig. 261 is a diagram showing the combined effect of the stresses, f' and f'', throughout the section, L M. The quadrilateral, K L M N, represents direct compression, and accordingly,

$$K L = f' = \frac{F}{b}.$$

The two triangles, L O P, M O R, represent respectively the compressive and tensile values of the bending moment; so

$$L P = \frac{F x p}{I} \text{ and } M R = \frac{F x (b-p)}{I}.$$

It will be noticed that, in the etched portion of the figure, the compression and tension more or less neutralise one another, and that at the point Q there is exact equilibrium. From Q to M tension predominates, and compression from Q to L. Calling the distance O Q, q, we may obtain its value by equating,

$$\frac{F}{b} = \frac{F x q}{I},$$

whence

$$q = \frac{I}{b x}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (53)$$

When it is undesirable to allow a tensile stress in any part of the section, as in the case of a curved gate built up of a series of vertical voussoirs, evidently the section must be so arranged that

$$\frac{F}{b} = \frac{F x (b-p)}{I},$$

whence

$$x = \frac{I}{b(b-p)}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (54)$$

This agrees with (53) when $q = b - p$, which is the condition for coincidence of the points M and Q.

Limits of Stress.—It is manifest that there is a limiting value for the stress intensity at L, beyond which it would be unsafe to compress the leaf without risk of collapse. Let us call this limiting value γ, and consider its relationship to the resultant pressure.

Substituting in (51), we have

$$\gamma = \frac{F}{b} + \frac{F\,x\,p}{I}.$$

For any rib of given dimensions, b, p, and I are fixed; in other words, they are constants, and the only variables are F and x.

Re-arranging, we get

$$F\left(\frac{I}{p\,b} + x\right) = \frac{\gamma\,I}{p}, \quad . \quad . \quad . \quad . \quad . \quad . \quad (55)$$

which is an equation of the form

$$F(C_1 + x) = C_2,$$

where C_1 and C_2 are constants. Now, this is the equation of a rectangular or equilateral hyperbola, one of whose asymptotes is the line L M, and the other is a line parallel to the longitudinal axis and at a distance,

$$C_1 = \frac{I}{p\,b},$$

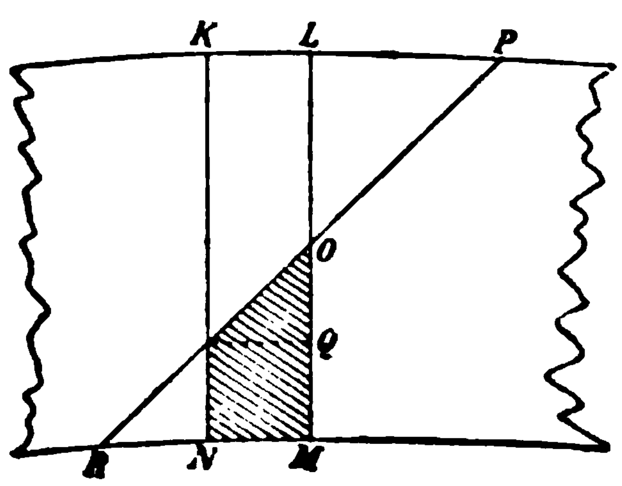


Fig. 261.

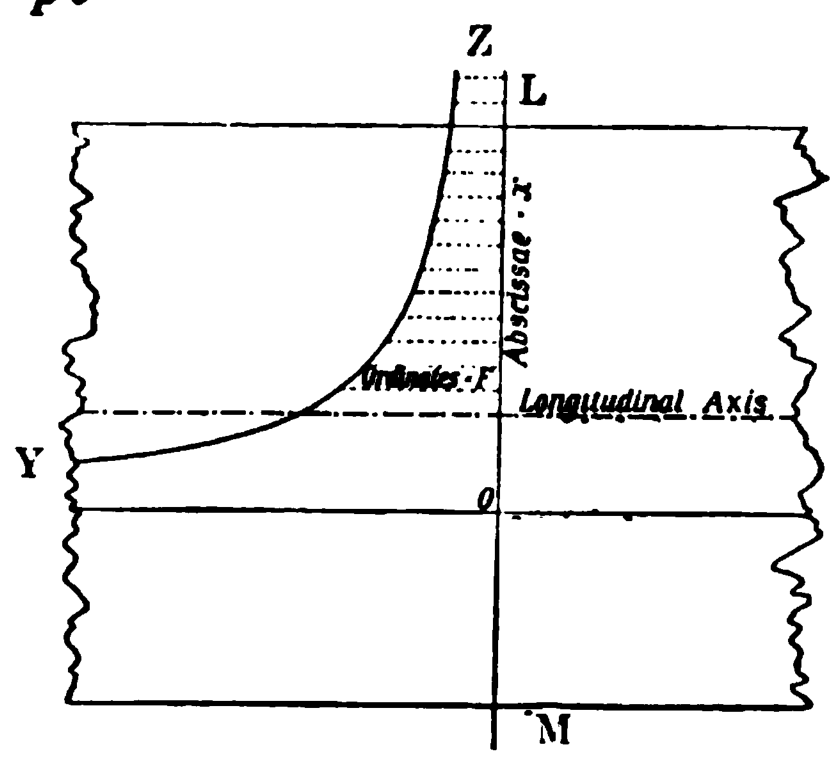


Fig. 262.

from it on the inner side. Consequently, if we fix upon a limiting value for γ, we may vary F and x within the range shown in fig. 262, where $\frac{I}{p\,b} + x$ constitutes the abscissa, and F the corresponding ordinate for any point of the hyperbolic curve, Y Z. The point O is the intersection of the asymptotes. Given the distance, x, from the longitudinal axis of the resultant, its maximum value is determined by the ordinate; and *vice versâ*, given the magnitude of the resultant, the extreme limit of its position may be deduced.

The diagram, fig. 262, is only applicable to resultants on the compressive side of the axis. For *loci* of F between O and M, it would be necessary to draw another hyperbola, with its origin on the other side of O; but instances of this kind do not usually occur in practice and need not be further considered.

Connecting Pieces.—Timber gates of the voussoir type are generally stiffened by horizontal connecting pieces (*vide* fig. 288) on the front of the gate, forming chords to the arc of the gate. The total moment of resistance in such cases is compounded of the separate moments due to the voussoir and the connecting piece, and since the angle of deflection producing the moment of resistance is the same in both members, it is evident that the distribution of stress due to bending will be similar, the amount and maximum intensity being determined by the relative breadths of the voussoir and connecting piece.

To draw the diagram of stress, find first the stress area, L P O R M (fig. 263), for the voussoir, considered as acting alone. Then through C the neutral axis of connecting piece draw Q S parallel to P R. The area M Q C S N represents the proportionate stress in the connecting piece. We must now reduce both areas in the same proportion until their sum is equal to the area of stress caused by the bending moment—that is, L P O R M. To do this, divide L P in the point X such that $LX : XP :: OL^2 : CM^2$. Draw X X′ through the point O and Y Y′ parallel to it through the point C. Then the etched portion, L X O X′ M Y C Y′ N, is the required stress diagram as regards bending moment only. The direct compressive stress is taken by the voussoir as before.

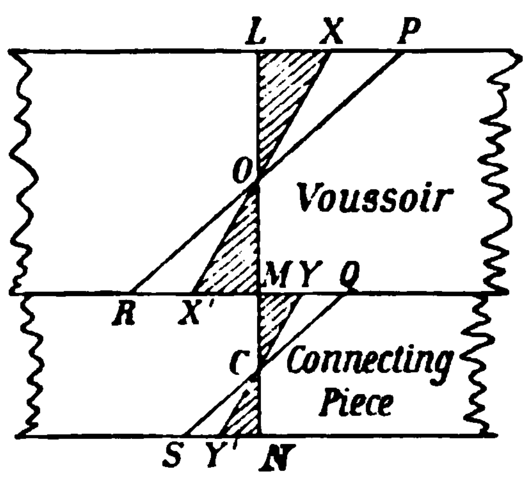


Fig. 263.

The proof of the diagram is as follows:—The triangles, O X P and C M Y, must be equal to fulfil the required conditions. Hence

$$XP \times LO = MY \times CM$$

$$MY = \frac{XP \times LO}{CM}.$$

Also, since X X′ and Y Y′ are to be parallel,

$$\begin{aligned} MY : CM &:: X'M : MO \\ &:: LX : LO \end{aligned}$$

$$MY = \frac{CM \times LX}{LO}.$$

Equating the two values of M Y,

$$\frac{XP \times LO}{CM} = \frac{CM \times LX}{LO},$$

$$\frac{XP}{LX} = \frac{CM^2}{LO^2}.$$

So delicate an adjustment of stress depends upon conditions which cannot be obtained in practice, and it is certainly advisable to construct the voussoirs of a gate so that they may be able to take the whole of the stress unaided by the connecting pieces.

Typical Examples—1. *Horizontal Rectangular Rib.*—In this case $p = \frac{b}{2}$; $I = \frac{b^3}{12}$, when the depth is unity; and F is the total stress, divided by the number of ribs.

The value for L P (fig. 261) is

$$\frac{F x p}{I} = \frac{6 F x}{b^2},$$

and the neutral axis of a rectangular beam being assumed to coincide with its horizontal axis of symmetry, the same value is equally applicable to M R.

When the resultant passes through the point L—*i.e.*, when $x = \frac{b}{2}$, the compression at L becomes $\frac{F}{b} + \frac{3F}{b} = \frac{4F}{b}$, or exactly four times the intensity which it has when the resultant passes through the point O.

2. *Vertical Voussoirs.*—Here it is necessary to avoid tension in any part of the joints, no duty being expected from the connecting bolts in this respect.

The point Q is given from (53) by

$$q = \frac{I}{b x} = \frac{b^2}{12 x},$$

and when q is made equal to $\frac{b}{2}$, which, as we have already demonstrated, is the limit consistent with the absence of tension, we have

$$\frac{b}{2} = \frac{b^2}{12 x},$$

so that

$$x = \frac{b}{6},$$

which signifies that the line of pressures must lie within the middle third of the thickness of the gate.

3. *Horizontal Iron Girders.*—Assuming the web horizontal and the flanges vertical, so that the plane of the resultant coincides with that of the web, supposed indefinitely thin, no difference exists between the formulæ in this case and those for rectangular ribs, beyond the complication introduced by the somewhat involved expressions for the value of the moment of inertia.

With flanges of equal area, symmetrically disposed about the centre of gravity of the section, the value of I may be conveniently expressed as the difference between the moments of inertia of two rectangles:—

$$I = \frac{d b^3}{12} - \frac{d_1 b_1^3}{12}, \quad . \quad . \quad . \quad . \quad . \quad . \quad (56)$$

where d_1 and b_1 are the dimensions of the combined side recesses of the irder.

The intensity of stress due to direct compression is

$$f' = \frac{F}{A} = \frac{F}{(d - d_1)(b - b_1)}, \quad . \quad . \quad . \quad (57)$$

and the maximum stress due to the bending moment is

$$f'' = \frac{F x p}{I} = \frac{6 F b x}{d b^3 - d_1 b_1^3}, \quad . \quad . \quad . \quad (58)$$

whence, combining, we obtain the maximum and minimum intensities in the outer and inner flanges respectively,

$$f = f' \pm f'' = F \left\{ \frac{1}{(d - d_1)(b - b_1)} \pm \frac{6 b x}{d b^3 - d_1 b_1^3} \right\}. \quad (59)$$

Another expression for the value of I, which neglects the thickness of the flanges and assumes their areas concentrated on a centre line in each case situated a distance, d, apart, with k as the area of the web, is

$$I = \frac{b^2}{2}\left(a + \frac{k}{6}\right). \quad . \quad . \quad . \quad . \quad (60)$$

If only an approximation be required, $\frac{k}{6}$ may be ignored as very small and

$$I = \frac{a b^2}{2}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (61)$$

When the cross-section of the girder is not symmetrical about the centre of gravity, the position of the latter may be obtained by dividing the depth of the girder inversely as the ratio of the flange areas. Thus, if a_1 be the area of the smaller flange and a_2 that of the larger, the distance of the centre of gravity from the larger flange will be

$$\frac{a_1}{a_1 + a_2} b, \quad . \quad . \quad . \quad . \quad . \quad (62)$$

where b is the horizontal dimension of the girder. Or it may be obtained graphically, thus:—Let A B (fig. 264) represent the web of the girder as a single line; set off horizontally A C = area of flange B, and B D = area of flange A. Join C D, and O is the required centre of gravity.

Using the notation of (62) a fairly approximate value for I is

$$I = a_1 b^2 \left\{ \frac{a_2}{a_1 + a_2} \right\}^2 + a_2 b^2 \left\{ \frac{a_1}{a_1 + a_2} \right\}^2 + \frac{k b^2}{12}. \quad . \quad . \quad (63)$$

For built girders, the moment of inertia will have to be calculated in detail from its component parts.

Gates with Vertical Co-planar Girders.—For straight or flat gates with discontinuous horizontal members, a different method of stress investigation is necessary. The system of co-planar vertical girders which derive no support from each other, such as contiguous vertical voussoirs do, involves, as has already been pointed out, the use of two horizontal transoms, one at the head and the other at the sill, to afford them the necessary support.

As the pressure increases with the depth, the total pressure, $\frac{w h^2}{2}$, is distributed unequally between these two members, in the proportion of 1 to 2, the amount being $\frac{w h^2}{6}$ at the top and $\frac{w h^2}{3}$ at the sill.

The verticals may accordingly be treated as independent beams, sustaining a uniformly increasing load. Under these conditions it is evident that the maximum bending moment cannot be at the centre. The bending moment at any point X may be found thus :—The pressure on the surface A X (fig. 265) of the gate is $\frac{w x^2}{2}$, acting at a point $\frac{x}{3}$ above X. Consequently the bending moment at X is

$$M_x = \frac{w h^2 x}{6} - \frac{w x^3}{6} = \frac{w x}{6}(h^2 - x^2). \quad . \quad . \quad . \quad (64)$$

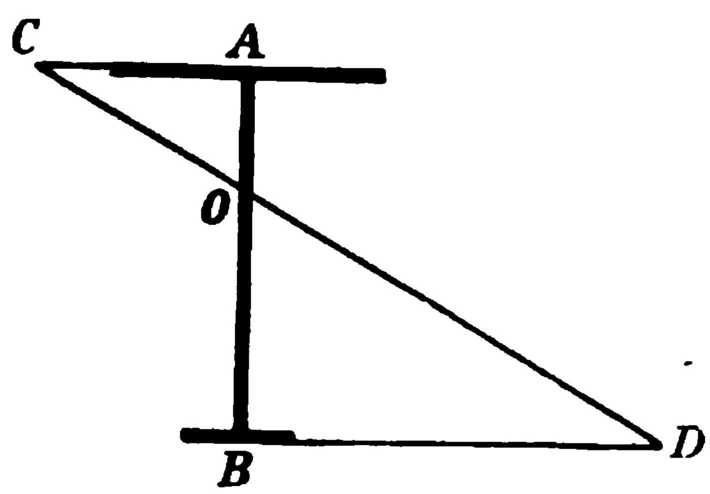


Fig. 264.

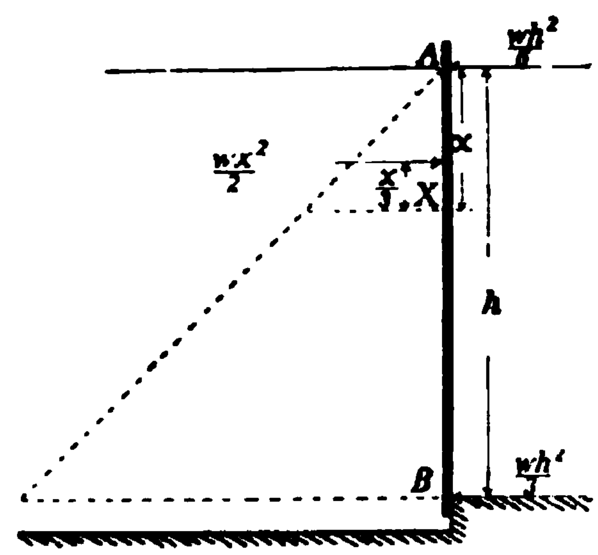


Fig. 265.

To obtain the maximum value of this expression it is only necessary to differentiate with respect to x and equate to zero.

$$\frac{d M_x}{d x} = \frac{d x (h^2 - x^2)}{d x} = h^2 - 3x^2 = 0$$

whence $\sqrt{3}\, x = h. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (65)$

In other words, the maximum bending moment is situated, not at the centre of pressure $\frac{h}{3}$, but at a point $\frac{h}{\sqrt{3}}$ below the surface of the water. The maximum bending moment at this point is

$$M_{max} = \frac{w h^3}{6\sqrt{3}} - \frac{w h^3}{18\sqrt{3}} = \frac{w h^3}{9\sqrt{3}}, \quad . \quad . \quad . \quad (66)$$

and the dimensions of the girder can easily be calculated by any of the methods applied to instances of beams under similar conditions of loading.

Subsidiary horizontal members are introduced between the verticals to transmit the pressure from the plating, which is made as thin as is consistent with durability and strength.

Stresses in Panels.—For all practical purposes the pressure on each unsupported area of plating or planking between the gate framing may be taken as uniform, an assumption which is, of course, to a certain extent,

erroneous. The variation from the truth is greatest in the case of the topmost panels, and the approach to accuracy increases with the depth. No account is taken, generally speaking, of the support derived from the fixture of the ends, nor, in cambered gates, of that due to the curvature, any excess of strength in these respects being set off against possible loss from corrosion or decay.

Calling the shorter unsupported length of the panel l, and d the depth of the centre of the panel below the surface, the maximum bending moment is $\frac{w\,d\,l^2}{8}$.

Then, if t be the thickness of the plate and f the safe maximum fibre stress, the moment of resistance is $\frac{f t^2}{6}$; and, equating,

$$\frac{f t^2}{6} = \frac{w\,d\,l^2}{8},$$

whence

$$t = l\sqrt{\frac{3}{4}\cdot\frac{w\,d}{f}}. \qquad (67)$$

In the foregoing expression the unit is the foot. It will be, perhaps, more convenient to express t and l in inches. Giving w its numerical value, the expression then becomes—

$$t = l\sqrt{\frac{d}{3f}}. \qquad (68)$$

Mr. Ivan C. Boobnoff, naval architect of the Imperial Russian Navy, proposes to calculate the thickness of plating for ships by a similar formula, deduced in a rather more elaborate manner—

$$t = l\sqrt{\frac{a}{4{\cdot}46 f}} \qquad (69)$$

These are theoretical thicknesses. There is in practice a minimum of ⅜ inch for iron and steel and 3 inches for wood, beyond which it is not safe to go, on account of the exceptionally rough usage to which the panels are subjected and their liability to corrosion and decay.

Practical Illustrations.—It will be useful at this stage to take an actual pair of gates and see how far their construction conforms to the theoretical requirements of the preceding formulæ. Examples of both wood and iron gates have been selected for this purpose, as representing two widely distinct types, the main dimensions of the entrances which they close being, as far as possible, alike, in order that a certain comparison may be instituted between them. For the plans and particulars relating to the metal gates the author is indebted to the courtesy of Mr. J. M. Moncrieff, of Messrs. Sandeman & Moncrieff, Newcastle-upon-Tyne.

Case 1.—Wooden Gates.—A pair of gates at Liverpool, each leaf consisting of a series of curved horizontal ribs, built in two voussoirs with connecting pieces, as shown by the drawings in figs. 266, 267, and 268. With the

exception of the two topmost connecting pieces and a rubber on the front of the gate, which are of pitchpine, the whole of the framing and panelling is of greenheart, fastened with galvanised iron bolts and straps.

The data for calculation are as follows :—

	Ft.	Ins.
Width of waterway, ...	60	0
Span of gates (between heel-post centres), .	63	8
Rise or versed sine of sill,*	10	0
Rise or versed sine of gates,* . . .	10	6
Radius of heel-post, ...	0	12
Length of leaf (water-bearing surface), . .	34	6
Distance from centre of heel-post to centre of meeting faces of mitre-posts, . . .	33	6

The gates are segmental in form, and so designed that the curve of pressure coincides with the back of the gate at the centre of each leaf. The thickness of the middle head is 2 feet.

The total height of each leaf is 34 feet 3 inches, of which 9 inches forms a sill abutment, leaving a height of 33 feet 6 inches capable of sustaining water pressure.

Adopting first the approximate formula (47) for the resultant, we obtain—

$$R = \frac{w h^2 l (l - \rho)}{4 r} = \frac{64 \times 33{\cdot}5 \times 33{\cdot}5 \times 34{\cdot}5 \times 33{\cdot}5}{4 \times 10{\cdot}5} = 1{,}976{,}442 \text{ lbs.}$$

This may be checked by drawing in the diagram of stresses as illustrated in fig. 256, from which it will be found that the radius of curvature is 55 feet. Hence

$$R = \frac{55 \times 64 \times 33{\cdot}5 \times 33{\cdot}5}{2} = 1{,}975{,}160 \text{ lbs.}$$

The agreement is very close. Let the result be taken in round numbers at 882 tons.

Now, the maximum bending moment is at the middle head, where the curve of pressure is situated 12 inches outside the longitudinal axis of the gate. At this point its direction is normal to the back of the gate, so there will be no shearing stress along the joints on either side of the middle head.

In the preliminary investigation it will be recollected that when the curve of pressure lay upon the outer edge of a horizontal rib the intensity of stress in the outermost fibres was found to be four times that of the simple compression due to a resultant acting along the gate axis. The compressive intensity is $\frac{882}{24}$ tons, the gate being 24 inches in thickness. Accordingly the maximum stress intensity is $\frac{882 \times 4}{24} = 147$ tons over the whole depth of the gate.

* The versed sine is measured in each case from the line through the centres of the heel-posts and extends to the point of the sill and the centre of the meeting faces of the mitre-posts respectively.

Taking the ultimate compressive stress of greenheart at 8·5 tons per square inch, it is evident that a minimum depth of some 18 inches of solid rib is needed to withstand the 147 tons compression. This, however, is the critical value, when the material is tested to breaking point, and as it is inadvisable to take a less factor of safety than 10, 180 inches, or 15 feet, in depth is actually required. As a matter of fact, in the gate in question the total depth of solid rib amounts to over 20 feet, so that the factor of safety adopted lies between 13 and 14—a by no means excessive value for a wooden gate, having regard to the duties which it is called upon to perform.

In the foregoing calculation no account has been taken of the connecting pieces, for reasons which have already been given. They can only be looked upon as affording a reserve of strength for contingencies.

Allowing a working stress of $1\frac{1}{2}$ tons in the outermost fibres of the greenheart planking, the thickness of the bottom panel is deduced from

$$t = 1{\cdot}25 \sqrt{\frac{27}{3 \times 1\frac{1}{2}}} = 3{\cdot}05 \text{ inches.}$$

This is an ample allowance, for it takes no account of the fixture of the ends, and as the stress on the other panels is much less, a uniform thickness of 3 inches has been adopted throughout.

Case II.—Steel Gates.—A pair of gates for a graving dock on the River Blyth, constructed in mild steel, with greenheart heel and mitreposts and clapping sill, as shown by the drawings in figs. 269 to 273.

Data:—

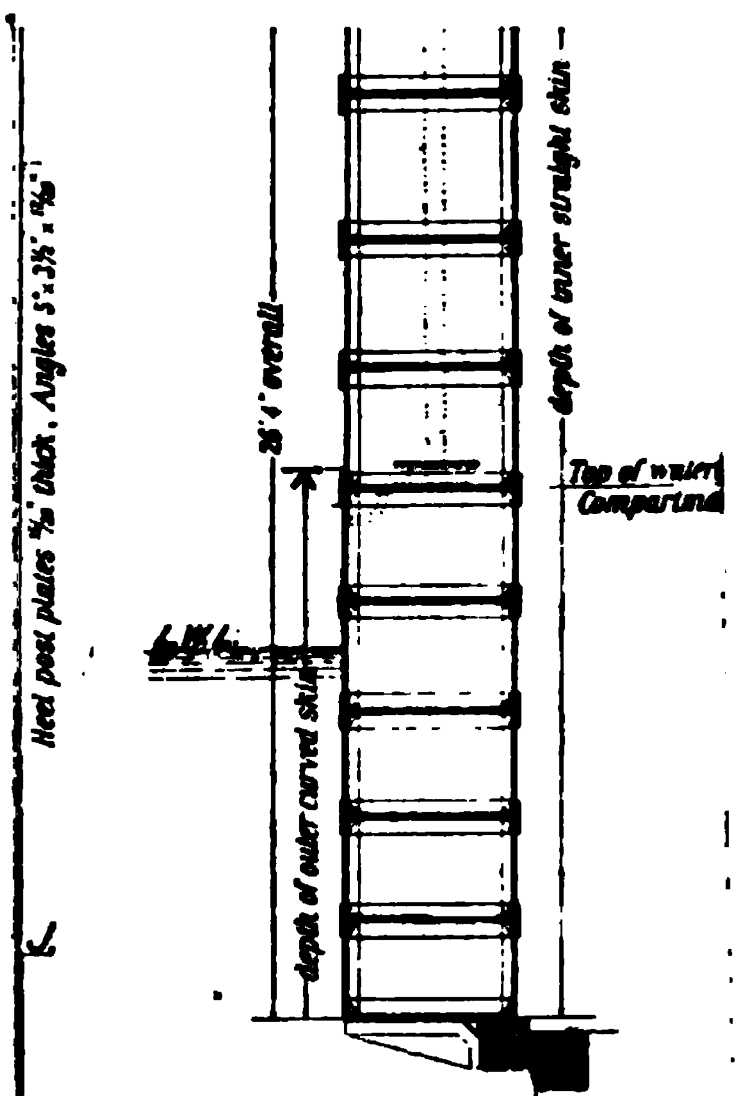
	Ft.	Ins.
Width of waterway, between fenders, . .	60	0
,, ,, between faces of walls, .	61	0
Span of gates (between heel-post centres), .	64	6
Rise of versed sine of gates,* . . .	10	9
Height of gate above dock sill, . . .	26	6
Depth of lowest compartment in body of gate, in order to allow ample room for rivetters, not less than	2	0
Width of leaf at each end, for reasons of access, ...	1	9
Width of leaf at centre, in order to ensure line of pressure passing within the gate, .	3	9

Dealing with the question of buoyancy in the first place, the horizontal sectional area of one leaf, as measured from plan, is about 108 square feet, so that the displacement, with the five lowermost compartments formed into a buoyancy chamber, is roughly,

$$\frac{11\frac{1}{12} \text{ feet depth} \times 108 \text{ square feet area} \times 64 \text{ lbs.}}{2{,}240} = 34\tfrac{1}{4} \text{ tons,}$$

while the actual weight of one leaf complete = $41\frac{1}{4}$ tons nearly, leaving

* See footnote, p. 333.



— Section at Vertical —
— Diaphragm —

7 tons as the nett positive weight keeping the leaf down, and preventing its rising off the heel pivot, when the gates are open and the water level is not lower than the top of the buoyancy chamber. When the gates are closed and the dock is pumped dry, the upward lifting effort of the water on each leaf exceeds $34\frac{1}{4}$ tons, owing to the fact that the clapping sill projects behind the body of the leaf, but even with the water level right up to the top of the gates, which would be a very remote contingency, the additional lifting force per leaf only amounts to about

$$\frac{9 \text{ square feet} \times 27\frac{1}{3} \text{ feet} \times 64 \text{ lbs.}}{2{,}240} = 7 \text{ tons nearly,}$$

making a total lifting force of $41\frac{1}{4}$ tons, or just equal to the weight of one leaf. But this lifting force could only exist when the gates were exerting an enormous thrust against the hollow quoin, and the friction would be amply sufficient to prevent the gates rising, even under such exceptional circumstances.

Owing to the water-bearing surface for the five lower compartments being the curved outer plating, while for the five upper compartments it is the straight inner plating, it is necessary to lay down two lines of pressure, and these are shown in fig. 273; but it will be noticed that they differ only very slightly from each other, and that the centres of the circular curves are practically coincident with a common radius of 53 feet 9 inches, say 54 feet. Also, because the ribs are not disposed according to zones of equal pressure, it will be necessary to treat each one separately, instead of dealing with the gate as a whole, as in the previous example. Space will not permit of our taking more than two cases, which, however, will be sufficient to indicate the method of dealing with the rest.

First take the rib at a depth of 24 feet $3\frac{1}{2}$ inches, and deal with the section 5 – 5. The section of the rib is shown in fig. 274.

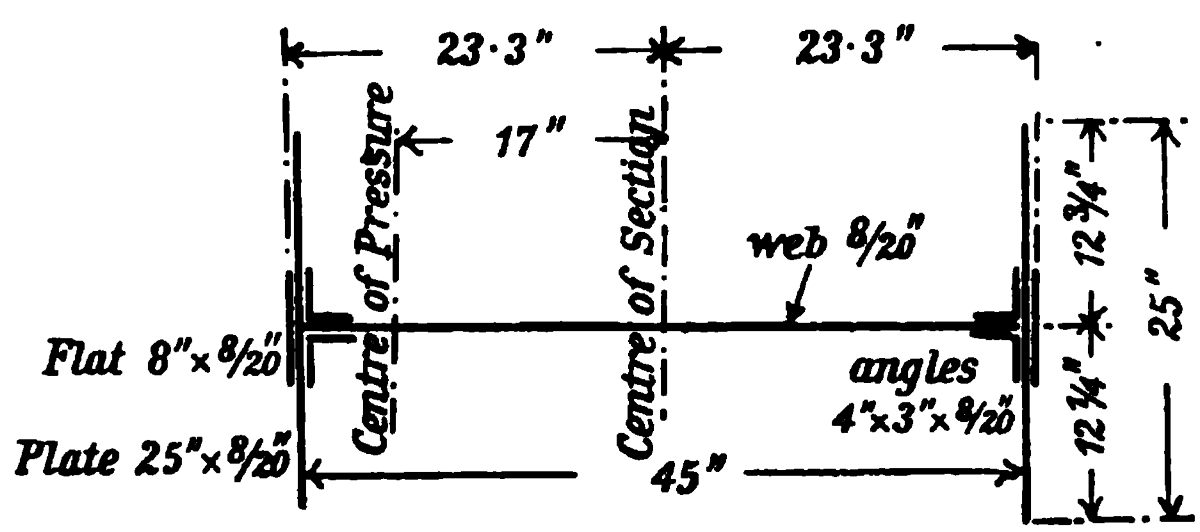


Fig. 274.

The water pressure on the face of the rib is

$$2{\cdot}08 \times 24{\cdot}3 \times 64 = 3{,}240 \text{ lbs., say.}$$

Hence, the resultant pressure:

$$R = 3{,}240 \times 54 = 175{,}000 \text{ lbs., nearly,}$$

and, since the direction of R is parallel to the centre of section, R = F.

The cross-sectional area is as follows:—

Web, . .	$45 \times \frac{8}{20}$	= 18	square inches.
Two plates, each,	$25 \times \frac{8}{20}$	= 20	"
" "	$8 \times \frac{8}{20}$	= 6·4	"
Four angles,	$4 \times 3 \times \frac{8}{20}$	= 10·56	"
		54·96	"

Say, 55 square inches. From formula (51) we have—it being noted that the flanges are symmetrical about the centre of section—

$$f = f' \pm f'' = \frac{F}{A} \pm \frac{F\,x\,p}{I}$$

$$= \frac{175{,}000}{55} \pm \frac{175{,}000 \times 17 \times 23{\cdot}3}{21{,}808}$$

$$= 3{,}182 \pm 3{,}178 \text{ lbs.,}$$

that is, 2·84 tons per square inch on the outer flange and 4 lbs. per square inch on the inner flange.

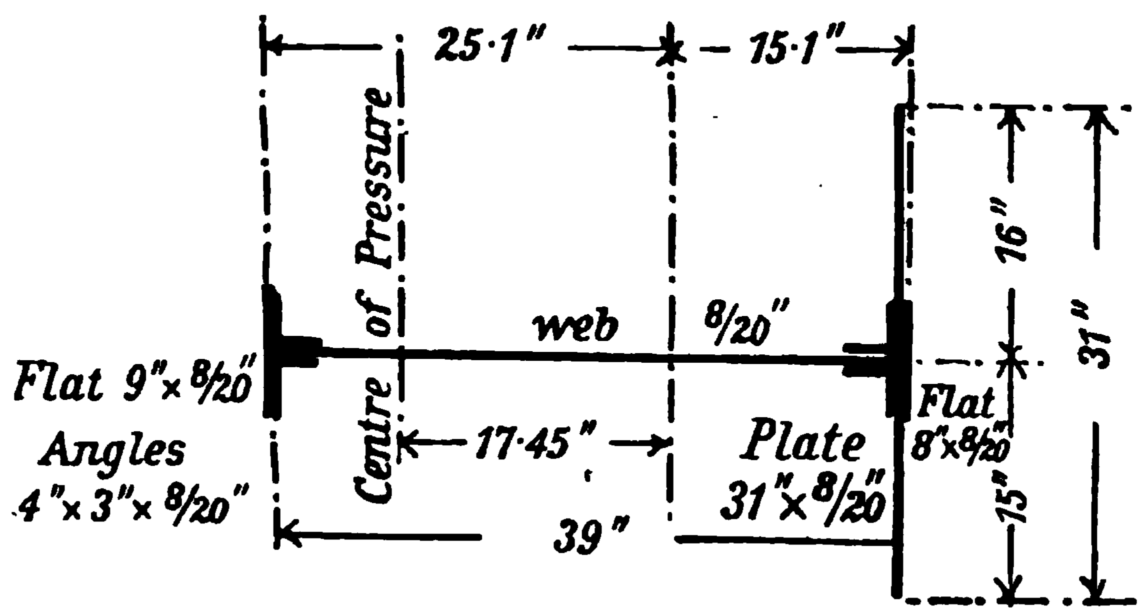


Fig. 275.

Now, take the rib at a depth of 12 feet $9\frac{1}{2}$ inches, and deal with section 3 – 3, shown in fig. 275. In this case the inner plating is the water-bearing surface, whose radius = 51 feet, nearly.

$$\text{Water pressure} = 2{\cdot}6 \times 12{\cdot}8 \times 64 = 2{,}130 \text{ lbs.,}$$

$$F = R = 2{,}130 \times 51 = 109{,}000 \text{ lbs.,}$$

in round numbers.

Cross-sectional area of rib—	$9 \times \frac{8}{20}$	= 3·6	square inches.
	$39 \times \frac{8}{20}$	= 15·6	"
	$31 \times \frac{8}{20}$	= 12·4	"
	$8 \times \frac{8}{20}$	= 3·2	"
Four angles,	$4 \times 3 \times \frac{8}{20}$	= 10·56	"
		45·36	

Say, 45 square inches. In this case the flanges are not symmetrical about

the centre of section. Accordingly, we must find the stress in each flange separately.

Outer Flange.

$$f = \frac{F}{A} + \frac{F x p}{I}$$

$$= \frac{109{,}000}{45} + \frac{109{,}000 \times 17{\cdot}45 \times 25{\cdot}1}{11{,}957}$$

$= 2{\cdot}86$ tons per square inch.

Inner Flange.

$$f = \frac{F}{A} - \frac{F x (b-p)}{I}$$

$$= \frac{109{,}000}{45} - \frac{109{,}000 \times 17{\cdot}45 \times 15{\cdot}1}{11{,}957}$$

$= 20$ lbs. per square inch.

It will be observed that all the foregoing stresses are well within the safe limits for mild steel.

The thickness of the lowermost plating works out as follows :—

$$t = l\sqrt{\frac{d}{3f}} = 15{\cdot}9\sqrt{\frac{25{\cdot}3}{3 \times 15{,}000}} = {\cdot}377 \text{ inch.}$$

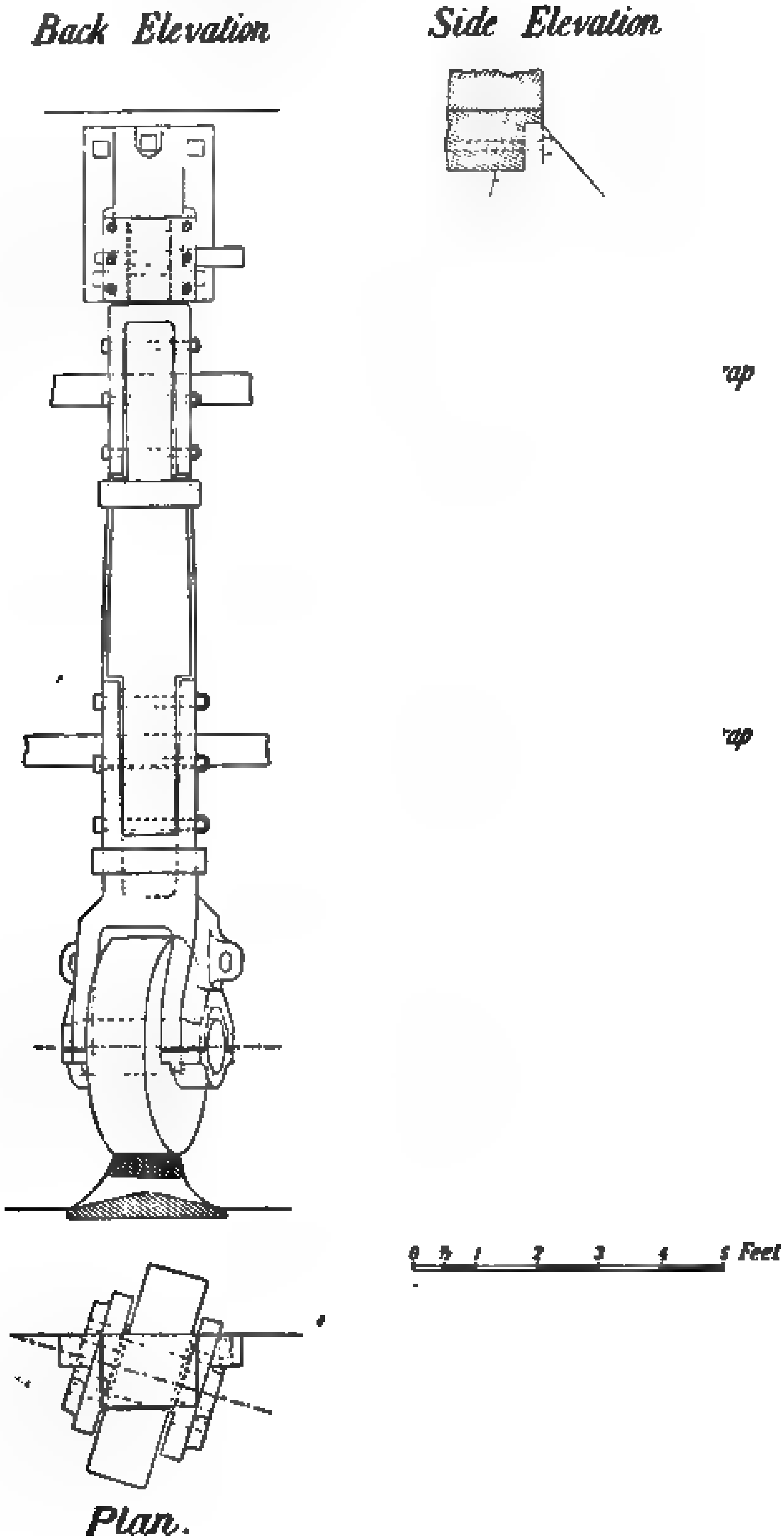
All the plates are actually made $\frac{8}{20}$ inch $= {\cdot}4$ inch.

Such is a very condensed outline of the calculations entailed in connection with the design of dock gates.

Gate Fittings.—We now turn our attention to some of the more prominent details connected with gate construction, leaving aside for the present those matters which relate to the working of the gates. These will be more advantageously dealt with in the chapter on Working Equipment.

Rollers and Roller Paths.—Gates may be entirely hung upon a pivot or axis at the heel-post, or they may derive partial support from truck wheels, or rollers, placed under them at one or more points. There is much conflict of opinion among engineers as to the value or otherwise of the latter method. On the one hand, it is urged that rollers add unnecessarily to the weight and expense of the gates, that they are liable to get out of order, that they are difficult to adjust and repair, and that, generally, they are a source of much anxiety and inconvenience. On the other hand, it is contended that they are a valuable means of support, that they reduce the friction on the heel-post and relieve the stress on the anchor blocks, and that they can be maintained in a state of efficiency with very little trouble. Generally speaking, Continental (more especially Dutch) practice inclines to the former view, English practice to the latter, but there is no absolute uniformity in either case. Rollers have been, and are being, dispensed with at Hull, while on the Mersey, the Manchester Ship Canal, and elsewhere they are still the invariable rule. It may, however, be fairly conceded that for small wooden gates and for iron gates with buoyancy chambers, rollers are not absolutely essential. Heavy wooden gates of large span certainly do gain in steadiness by the attachment of rollers to their outer extremities. Types of rollers in use at various ports are illustrated in figs. 276 to 278, 279, and 294.

Clapping Sills.—The facing of the lowermost horizontal member of a gate, forming a watertight joint with the dock sill, is almost invariably of wood, in wood and iron gates alike. Indiarubber, as a watertight material,



Figs. 276, 277, and 278.—Gate Rollers at Liverpool.

is not employed to any noticeable extent, though there is no apparent objection to its more extended use. An arrangement proposed by

M. Barret, dock engineer at Marseilles, in 1879, is illustrated in fig. 280. It consists of a buffer of plaited hemp covered with leather, with a

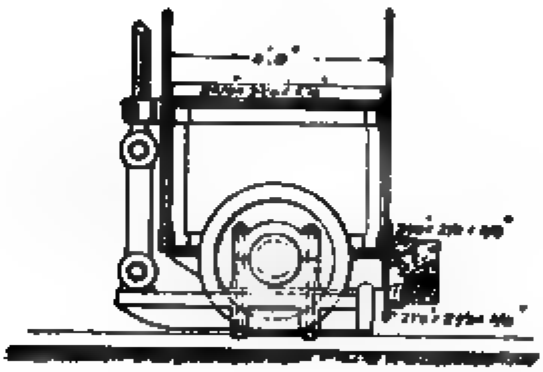


Fig. 279.—Gate Roller at Dublin.

Fig. 280.

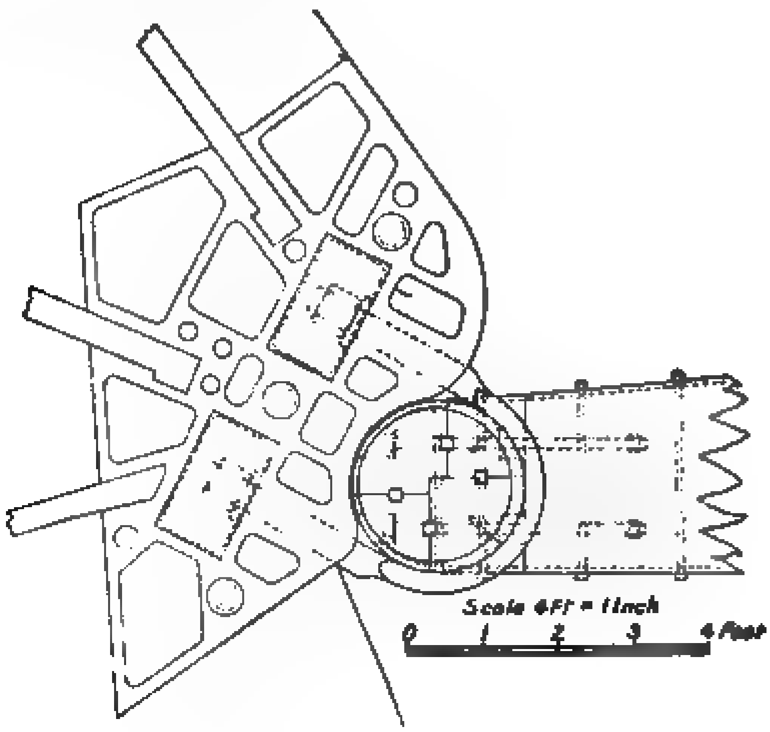


Fig. 281.—Gate Anchorage at Liverpool.

Fig. 282.—Gate Anchorage on the Tyne.

Fig. 283.—Gate Anchorage at Dublin.

pendant to absorb any play between the lower part of the gate section and the sill.

Sluices.—Sluices for levelling the water on both sides of a pair of lock gates preparatory to opening them, may be fitted in the gates themselves, alternatively to locating them in the side walls. The arrangement, however, has the disadvantage of adding very considerably to the weight of the gates, by reason of the apparatus required for opening and closing the sluices. The question is discussed somewhat more fully in Chap. vi.

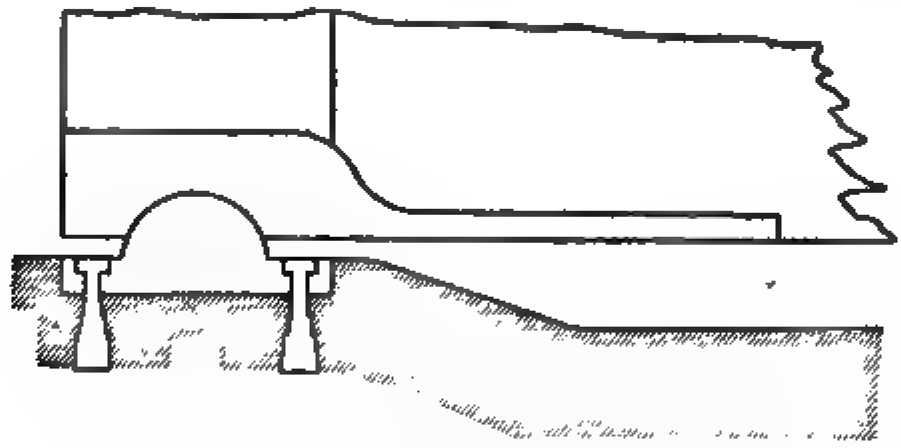


Fig. 284.—Gate Footstep.

Platforms.—Gates are usually fitted with a gangway at or about coping level. It is usually carried on brackets fixed to the topmost member of the gate. The handrail or chain guard should be removable, in order to facilitate the passing of warps and ropes when the passage is open.

Anchorage.—The top of the heel-post, or the upper pintle of a gate, revolves in a horizontal collar, bolted to and forming part of a suitable heavy casting, known as the anchor block, from which tie-rods or bars radiate to some distance, their ends being bedded in massive masonry. Several types of anchorage are shown in figs. 281, 282, and 283.

Footsteps.—The lower end of the heel-post may either be arranged as a pintle, fitting into a circular socket, or it may be fitted with a hollow casting to revolve upon a spherical surface. The latter arrangement is illustrated in two forms in figs. 284 and 285. The first of these is more suitable for small gates. In the second example, the play between the cylindrical pivot and the sides of the lower casting, or cup, is designed to allow of a slight clearance between the heel-post and the hollow quoin, during the movement of the former, so as to diminish the friction. A

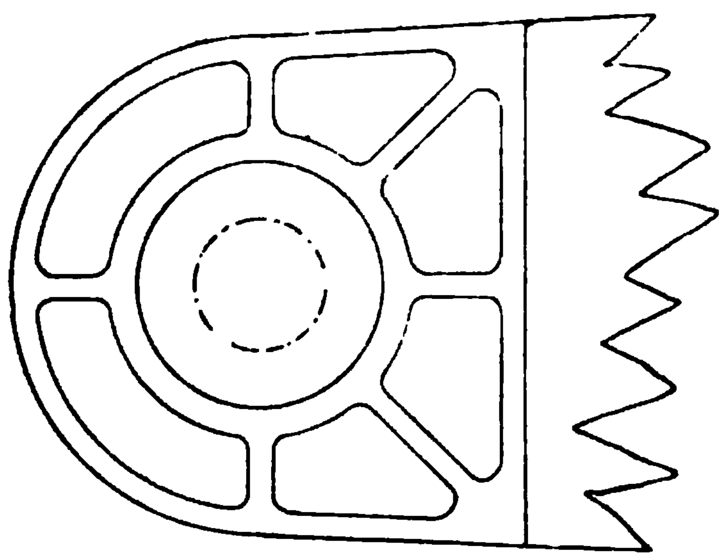
Sectional Elevation.

Fig. 285.—Gate Footstep.

suitable composition for the bronze alloys of the various parts as adopted at Liverpool is as follows:—

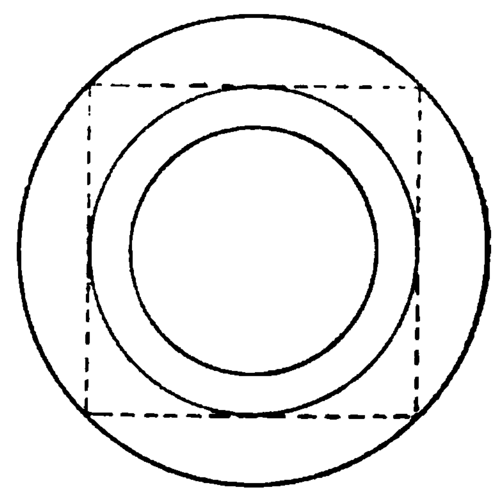
	Copper.	Tin.	Zinc.
Pivot, ...	16 ozs.	$2\frac{1}{2}$ ozs.	...
Ball, ...	16 „	$3\frac{1}{2}$ „	...
Heel-hoop, ...	16 „	2 „	$\frac{1}{2}$ oz.

The addition of a small portion of phosphorus is said to have the effect of preserving the surface of the metal from corrosion. Aluminium bronze, containing 90 parts of copper to 10 of aluminium, is a very strong alloy, which does not readily corrode, but it is very expensive. Manganese bronze is another compound possessing equal durability and strength. Steel is quite out of the question. It is speedily destroyed by the salt water.



Plan of Base of heelpost inverted.

Fig. 286.



Plan of Cup.

Fig. 287.

Check chains from the mitre-post to volute or other suitable spring at the square quoin of the gate recesses, are a useful means of checking the impetus of a gate at the sill and preventing distortion.

Examples of Dock Gates.

It only remains to conclude this section with a few selected examples of wood and iron gates.

Liverpool is *par excellence* the port of wooden gates. Throughout the vast system controlled by the Mersey Docks and Harbour Board, there is not a single iron gate in existence up to the time of writing. Several of the locks and entrances are no less than 100 feet in width, but they are all fitted with wooden gates. A pair of gates closing a 60-foot entrance has already been treated. By way of exemplification of the larger structures, the plan and vertical section of a leaf of the Canada Lock gates are shown in figs. 288 and 289. The timber is greenheart, connected by galvanised iron straps and bolts.

The principal feature of the gates along the great waterway leading to the **Port of Manchester** is their great solidity. Perhaps this is also their most essential requirement, for several serious accidents have already taken place in connection with them. For example, a few years back a steamer, improperly controlled, ran into a pair of gates and drove one leaf completely

Centre Line of Track.

Axle of

Roller.

Fig. 288.—Dock Gate at Liverpool (Sectional Plan).

Centre Line of Track.

Fig. 289.—Dock Gate at Liverpool (Vertical Section).

Scale 8 Ft = 1 Inch.
0 2 4 6 8 Feet

over the sill by shearing the mitre-post. Such an accident is, fortunately, exceptional, but demonstrates the possibilities which have to be contended

SECTION ON A.A SECTION ON B.B. PLAN.

Figs. 297, 298, and 299.—Dock Gates at Hull.

with. Figs. 290 to 296 are plans, sections, and elevation of a pair of 80-foot gates, constructed in greenheart, with iron straps and bolts.*

* Williams, Eliot, and Meade-King on "The Manchester Ship Canal," *Min. Proc. Inst. C.E.*, vol. cxxxi.

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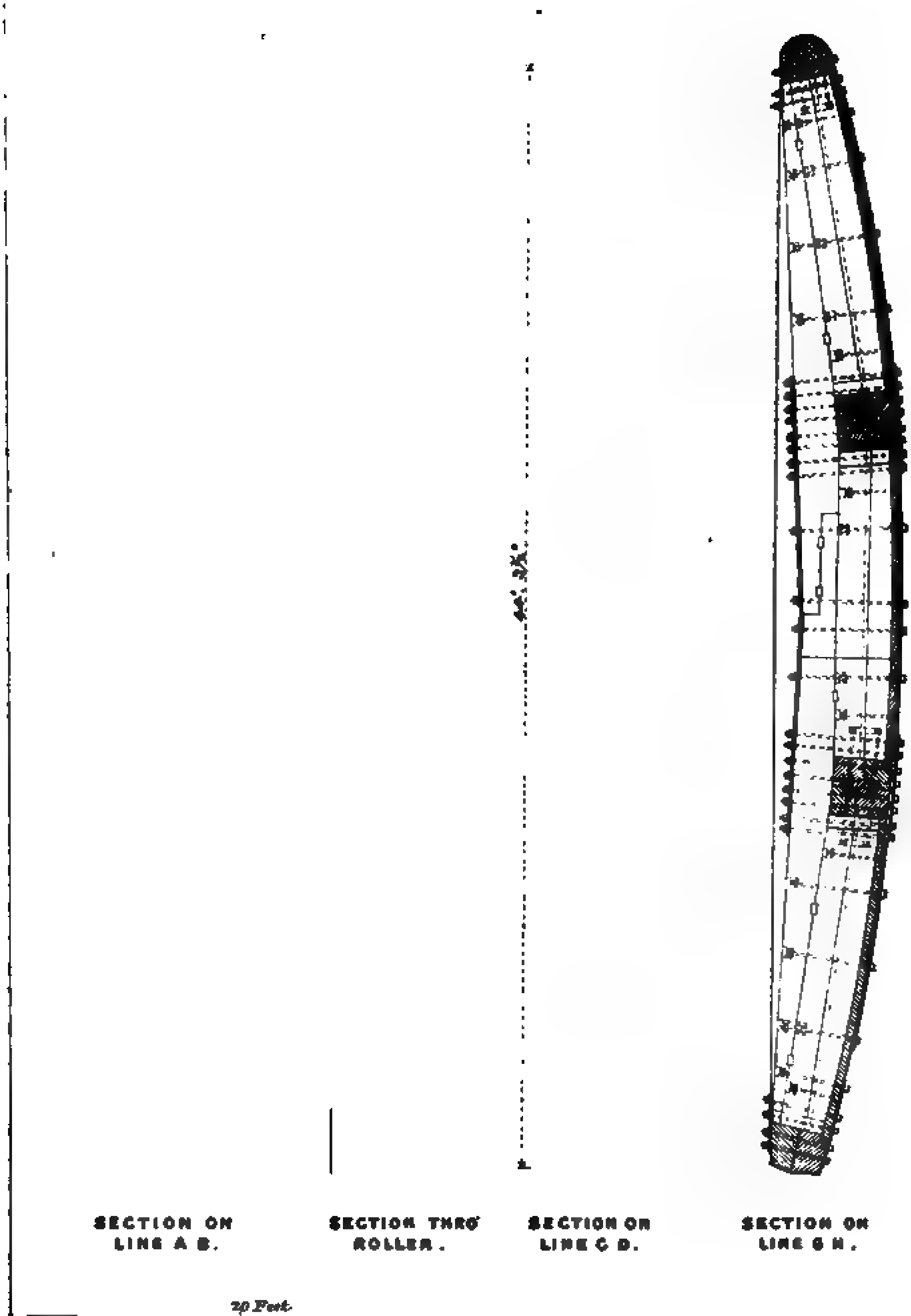


Gates.

Figs. 297, 298, and 299 illustrate the outer lock gates of the Alexandra Dock at **Hull**.* Each leaf consists of three framed greenheart voussoirs, similar in design to the Liverpool gates.

In all the preceding cases the wood panels are framed in between the ribs. In the diagrams of the gates at the North and South Locks, **Buenos Ayres** † (figs. 300 to 303), it will be observed that the sheeting is continuous throughout the height of the gates.

The iron gates at **Kidderpur Docks**, Calcutta,‡ are shown in figs. 304 to 309. The heel and mitre-posts are of greenheart.

A particular interest attaches to the pair of metal gates exhibited in figs. 310 to 312, as indicating an extremely ingenious device for overcoming a natural difficulty. The gates close the entrance to a graving dock on the **River Tyne**.§ The longitudinal axes of dock and river meet at an acute angle (fig. 313). Had the gates been constructed in the ordinary manner, with symmetrical leaves, and the line through the heel-post centres perpendicular to the axis of the dock, a considerable length of one quay would have been excluded from the graving dock, which would have been much shorter in consequence. By adopting the form of two unequal leaves, the designer, Mr. J. M. Moncrieff, has been enabled to utilise the axial length to its fullest extent. The dock entrance is 49 feet wide, and the lengths of the leaves are $41\frac{1}{2}$ and $22\frac{1}{4}$ feet respectively, their chord forming an angle of about 12° 35′ with the normal to the dock centre line. Under this arrangement the heel-post of the larger gate turns through a greater angle than is usually the case, and the hollow quoin has been purposely kept narrow to enable the end of the leaf to clear it. The inner and outer faces of the shorter leaf are concentric throughout, but the longer leaf needed the stiffening which could only be afforded by increasing the thickness or width at the centre, and accordingly a flatter curve has been adopted for the inner face.

A pair of iron gates at **Dunkirk**|| (figs. 314 to 319) are included as an example of the type of vertical girders. They have flat, parallel faces, and bear against a pointed sill. The extreme length of each leaf is 38 feet 4 inches, and the entrance closed is 69 feet wide. The plating covers the whole of the outer and the lower half of the inner face, forming a watertight chamber there. In a later type of gate erected at the same port the arrangement of the watertight compartments is slightly modified, as shown in the line diagrams, figs. 320 and 321.

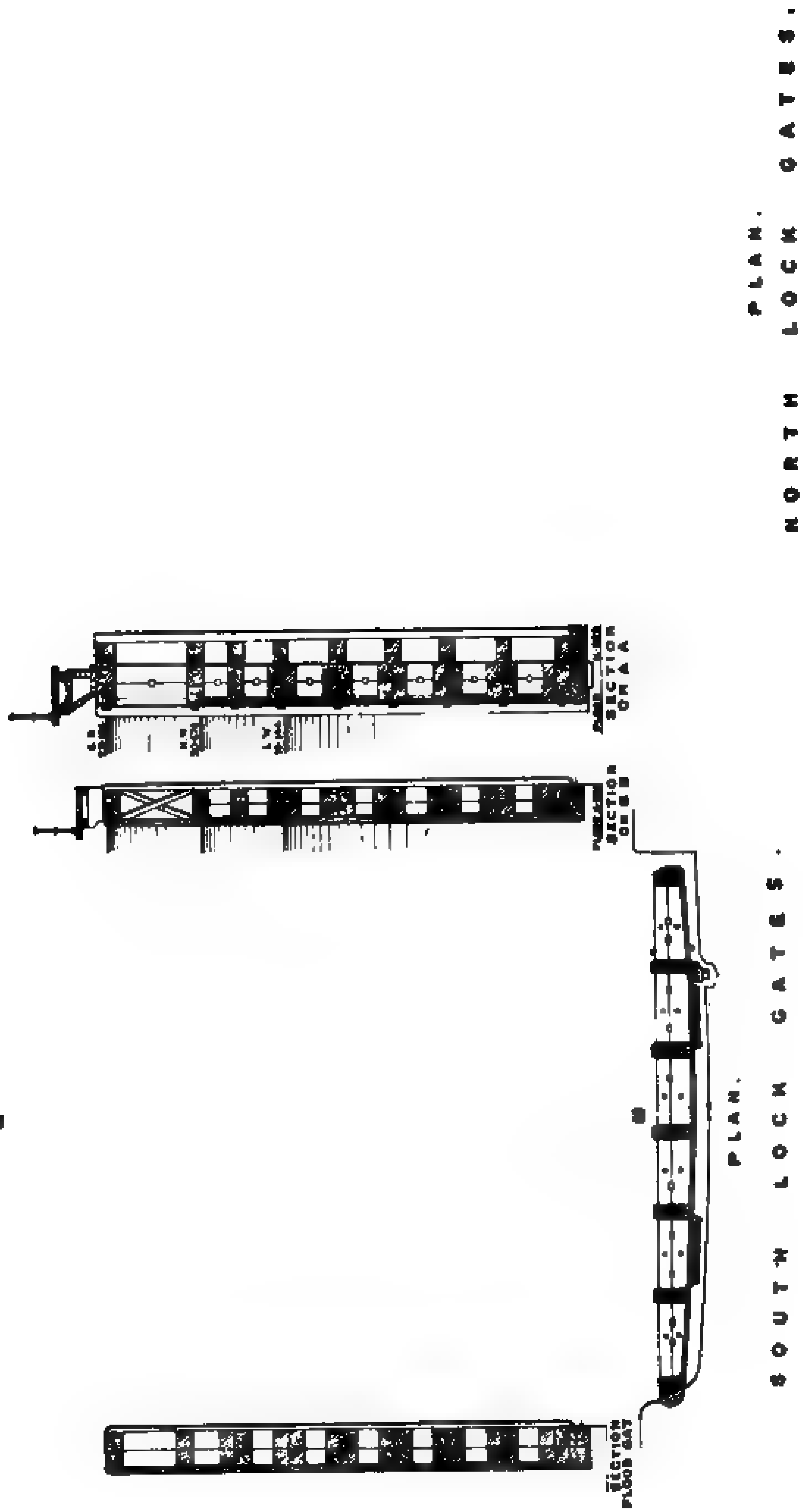
* Hurtzig on "The Alexandra Dock, Hull," *Min. Proc. Inst. C.E.*, vol. xcii.

† Dobson on "Buenos Ayres Harbour Works," *Min. Proc. Inst. C.E.*, vol. cxxxviii.

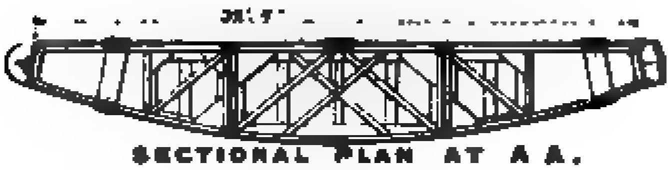
‡ Bruce on "The Kidderpur Docks, Calcutta," *Min. Proc. Inst. C.E.*, vol. cxxi.

§ Moncrieff on "Dock Gates of Iron and Steel," *Min. Proc. Inst. C.E.*, vol. cxvii.

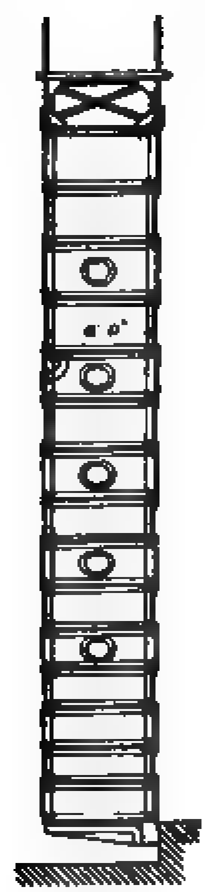
|| *Vide* Discussion on "Dock Gates," *Min. Proc. Inst. C.E.*, vol. lix.



Figs. 300, 301, 302, and 303.—Lock Gates at Buenos Ayres.



SECTIONAL PLAN AT A A.



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Figs. 304, 305, 306, 307, 308, and 309.—Dock Gates at Calcutta.

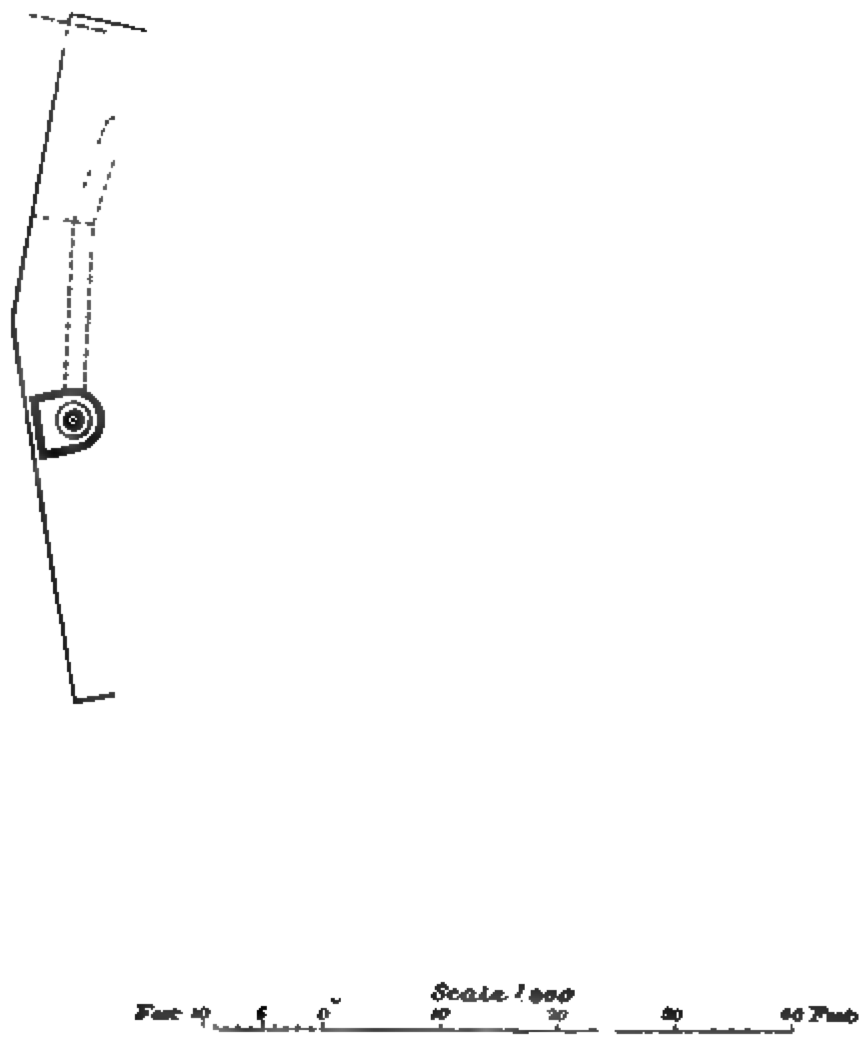

Fig. 313.—Graving Dock Entrance, River Tyne.

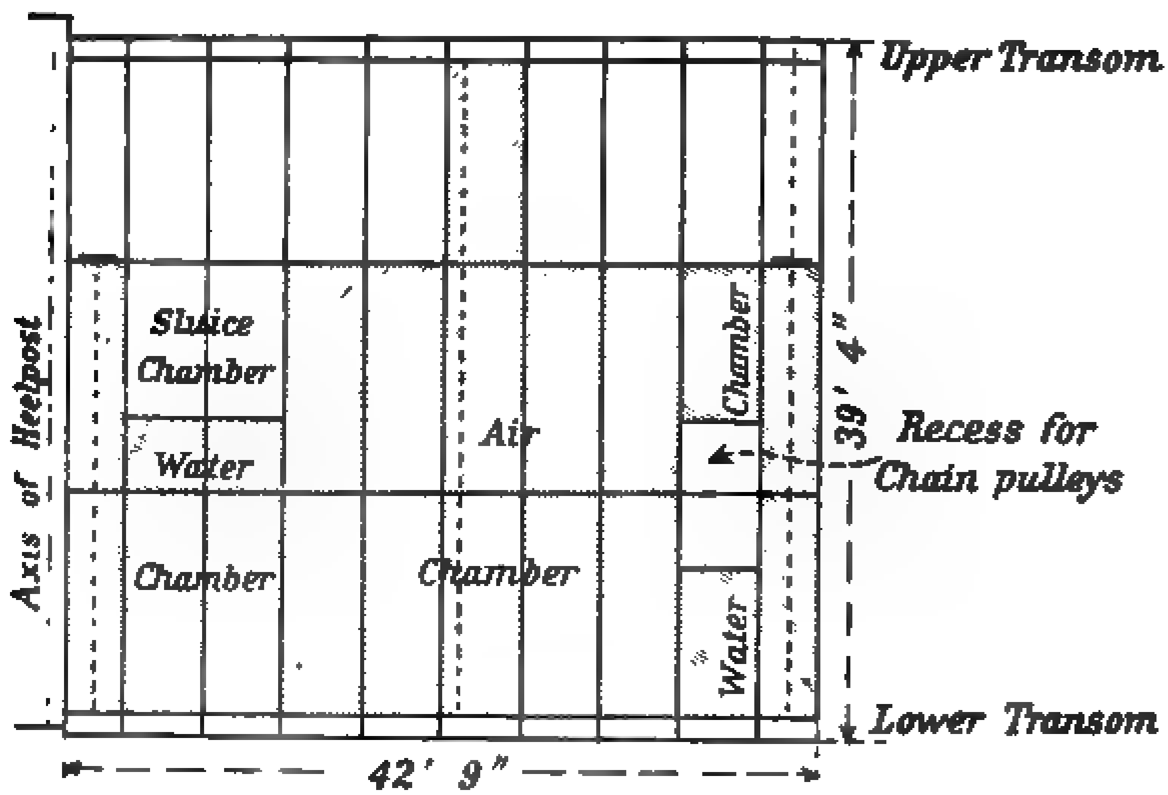

Fig. 320.—Elevation of Dock Gate at Dunkirk.

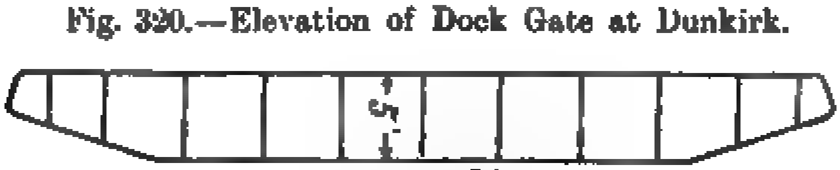

Fig. 321.—Plan.

The annexed table affords some statistics relating to different types of gate, collected from various sources. The writer is indebted in many cases to engineers at the several ports for the information.

E XV.

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and 319.

TABLE XXV.—Dock Gates.

Port or Locality.	Entrance or Passage.	Material.	Date.	Width of Waterway.	Depth of Water at H.W.O.S.T.	Height of Gates.	Weight of One Leaf.	Rise of Sill.	Cost per Sq. Foot of Gate.	Cost per Sq. Foot of Waterway.	Remarks.
			Circa	Feet.	Feet.	Feet.	Tons.	Feet.	£	£	
Hull,	Alexandra Lock,	Greenheart,	1883	85	32·5 / 34	38·7	170 / 176	14	48	53·7	3 pairs of gates, 2 outer pairs deeper than inner pair.
Blyth,	Graving Dock,	Steel,	1897	61	22·5	26·5	41·25	10	17·72	23·25	
Jarrow,	,,	Wrought iron,	1892	50	20	24	28·62	9·5	22·45	30·76	
South Shields,	,,	,,	1891	49	21	25·6	31·2 / 16·8	13·75	15·7	24·7	2 unequal leaves; 41·5 and 22·25 ft. long respectively.
Dublin,	,,	,,	1857	70	18·25	22·5	45	18	46·75	54·08	
Liverpool,	Langton Dock,	Greenheart,	1880	65	31	34·7	..	10·75	46·92	58·08	
,,	,, Branch Dock,	,,	1880	60	31	32·7	..	10	46·92	56	
,,	,, Graving Dock,	,,	1880	60	25	33	..	10	44·5	51·5	
,,	Huskisson Dock,	,,	1896	90	39·5	44·25	165	16	45·8	51·8	
,,	Canada Graving Dock,	,,	1890	94	32	39·75		16	47·2	53·7	Gates widened and deepened before completion.
,,	,, Dock,	,,	1895	90	33	37·75	153	15	42·16	50	
,,	Coburg Dock,	,,	1900	100	30·5	39·5	..	16·5	32·6	37·7	
London,	Greenland Lock,	Steel,	1898	80	30·25	38·2	185	20	..	..	
,,	Tilbury Lock,	Wrought iron,	1886	80	42 / 36	44·7 / 38·7	165 / 146·5	20	39	62	3 pairs of gates; outer and middle pairs deeper than inner pair.
,,	Blackwall Entrance,	Steel frames, wrought-iron skin,	1894	60	30	31	78	11·78	35·6	42·2	
,,	S.-W. India Dock,	Wrought iron and greenheart,	1809	55	27	30·75	..	14	.	54	
,,	Gallion's Entrance,	Steel frames, wrought-iron skins,	1886	80	36	40·33	125	15·5	37·2	48·6	
,,	Shadwell Entrance,	Wrought iron,	1855	60	28	.	70	..	.	..	
Avonmouth,	Lock,	Greenheart, pitchpine, and Memel,	1877	70	38·5	48	102	16·5	28·16	33·25	
Limerick,	Dock,	Steel,	..	70	23·5	26	45	17·5	25·3	30·75	
Newport, Mon.,	Graving Dock,	Oak and pitchpine,	1890	48	21·75	29·75	..	10	..	..	
Dunkirk,	North Lock,	Steel and galvanised iron,	1893	82	35·75	39·5	153	14·08	49	55·4	
,,	West Lock,	Galvanised iron,	1880	69	24·5	27·75	54	12·2	24	28	
,,	Barrage Lock,	,,	1887	69	20·84	24·06	44	12·2	18	21	
,,	Citadel Lock,	Oak and teak,	1883	42·7	20·34	23·75	30	7·84	23	27·4	
Harburg,	Harbour,	Oak,	1881	32·6	..	22·75		..	..	17·6	
,,	,,	Wrought iron,	1880	55	.	31	43	..	..	32	
Glückstadt,	Dock,	Oak,	1891	45	..	21	..	..	..	17·7	Price of oak, 7s. per cub. ft.
,,	,,	,,	1898	45	..	21	.	..	..	14·6	,, 5s. ,,
Emden,	Railway Dock,	Wrought iron,	1891	34·25		16	..	..	..	18	Now out of use.
Bremerhaven,	New Harbour,	Cast and wrought iron,	1852	72	.	38·2	..	..	..	25	Heel-post cast iron, re- [illegible]
,,	,,	Steel,	1900	72	.	38·2	61	..	..	36·4	
Kaiser William Canal,	Lock,	,,	1894	82	..	51·5	127	..	..	24	
,, ,,	,,	,,	1894	82		37	93	..	..	24·8	
,, ,,	,,	,,	1894	82	..	43	99	..	..	22·4	Horizontal girders.
Buenos Ayres,	South Lock,	Greenheart,	1898	65·6	26·75	35	103	..	..	..	Width at coping level.
,,	North Lock,	,,	1898	66·6	27	35	150	..	..	..	,, ,,

Dock Caissons.

The primary meaning of the word caisson appears to be a box or chest (Fr. *caisse*), but its use has been extended, in maritime engineering, to include all hollow structures, not being gates, used to close entrances or passages. Generally speaking, though not universally so, the horizontal axis of a caisson is a straight line, differing in this respect from gates, the leaves of which usually meet at an angle. Any absolute distinction, however, between gates and caissons is difficult to draw, owing to the extreme variety of types in both classes.

Stresses in Caissons.—The stresses induced in a horizontally-framed caisson *in situ* are those incurred by a series of beams uniformly loaded and supported at each end. It is only necessary to find the proportion of hydrostatic pressure on each beam and then to consider it as a uniformly distributed load. The bending moment at the centre will be $\frac{w\,l^2}{8}$, where l is the length of the unsupported portion. The moment of resistance will be $a f d$, where f is the maximum permissible unit-stress in either flange, with area a, and d the distance between the centres of gravity of the flanges. Equating the two moments, and noting that a and d are the only variables, we get

$$a\,d = \frac{w\,l^2}{8f},$$

in which, by selecting a value for the width of the caisson, we determine the corresponding sectional area in the flanges of the horizontal beams of which it is composed.

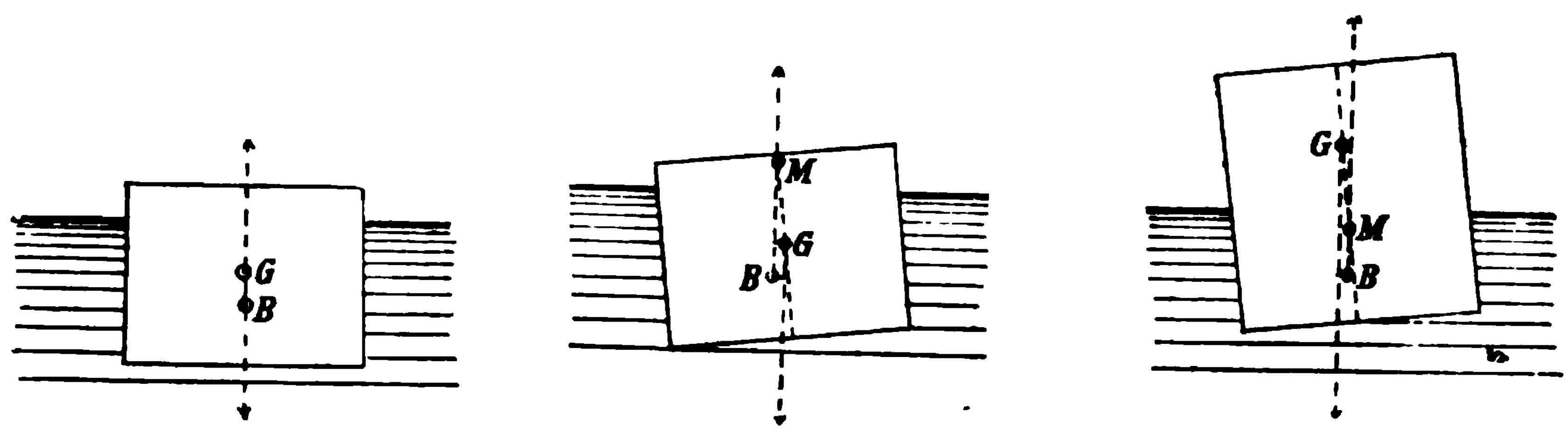
Where the horizontal members are, however, discontinuous, and the external thrust is sustained by a series of verticals, it will be necessary to provide a substantial transom at the top capable of taking one-third of the total pressure. The distribution of stress in this case has already been investigated in connection with gates constructed on identical lines.

Apart from the resistance of a caisson to lateral pressure, it is furthermore necessary to take into account its conditions of stability under the upward pressure of the water. This upward pressure, which is equal in amount to the weight of the volume of water displaced by the caisson, may cause its derangement and ultimate capsisal, if not properly provided for.

In every floating body, there are two points which determine the stability of its position. One is the centre of gravity (G, fig. 322) of the body itself, and the other, the centre of gravity (B) of the fluid displaced, otherwise designated the centre of buoyancy. These may have any number of positions relative to one another, but as long as they remain in the same vertical line the equilibrium is complete. If, however, after a slight displacement of the body, the points are no longer in the same vertical, it is manifest that there is a couple, $W\,x$ (W = weight of displaced fluid;

x = horizontal distance between verticals), tending either to restore the body to its former position, or to completely overturn it.

In fig. 323 a floating body is represented as having undergone a slight displacement. The centres of gravity and buoyancy now occupy relatively different positions, unless the body be a homogeneous sphere. Assuming that it is not, if the centre of gravity lie below the centre of buoyancy, the couple is clearly a righting one. If, on the other hand, the centre of buoyancy lie below the centre of gravity, the couple will not necessarily be an overturning one; its effect will depend upon the following condition. Premising that the point, in which the vertical through the centre of buoyancy after a slight displacement intersects the vertical through the centre of buoyancy in its former position of equilibrium, is designated the *metacentre*, the condition for the restoration of equilibrium is that the metacentre shall lie above the centre of gravity of the body, otherwise the latter will tend to depart still further from the position of equilibrium. The two effects are illustrated in figs. 323 and 324.



Figs. 322, 323, and 324.

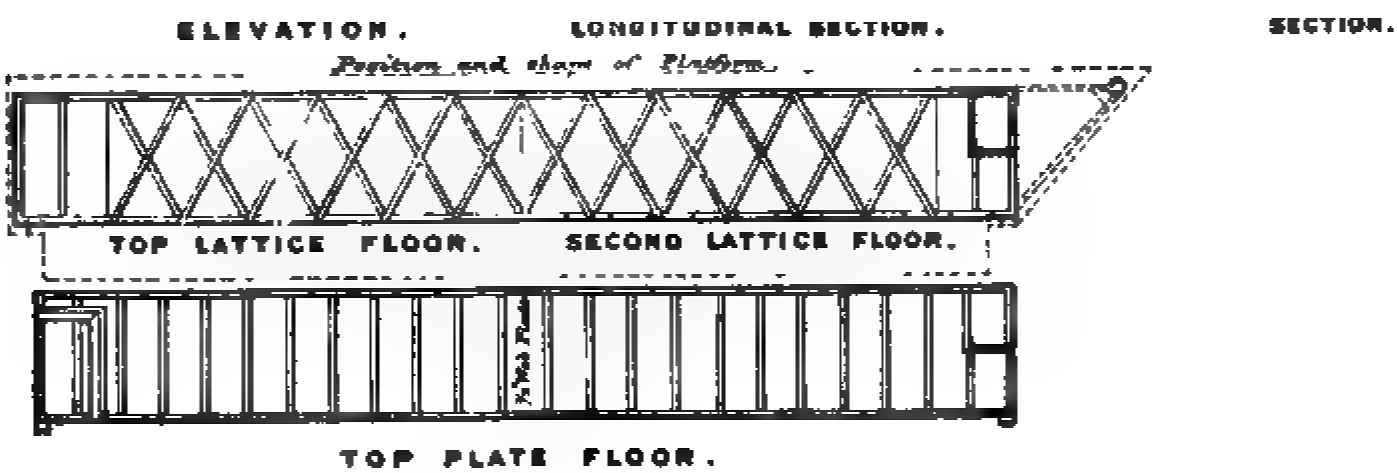
In the case of caissons, it is particularly desirable that the metacentre should be well above the centre of gravity, say not less than 2 to 3 feet, but the stability of the caisson will be more completely assured by ballasting it until the centre of gravity falls below the centre of buoyancy. A margin of 18 inches or so will be found sufficient for safe working. If the caisson be fitted with air chambers and a tidal deck, it will certainly be advisable, if not imperative, to adopt the latter precaution.

Classification of Caissons.—Caissons are of very diverse design, but they admit of a broad classification into swinging, traversing, and ship caissons.

Swinging Caissons have already been referred to, under the name of gate caissons, as forming an intermediate class possessing characteristics common to both gates and caissons. Like the former, they turn or swing upon a vertical axis fixed at one side of a waterway, and they have all the drawbacks attaching to a single leaf gate, in regard to the excessive length of side recess required for their accommodation when out of use. On the other hand, they are built with much broader beam than gates, and this gives them the compensating advantage of a wide roadway for traffic at quay level, which would otherwise be impracticable without the aid of a special swing bridge. This feature, however, is more or less common

to all classes of caisson. Swinging caissons are not numerous. One is chosen for illustration from the entrance to a graving dock, leading out of the Victoria Dock at Dundee (fig. 325). In plan, its only distinguishing feature from a caisson of the ordinary rectangular type is the hinge about which it turns, which is situated at the apex of a triangular arm. One side of the arm forms a continuation of the outer face of the caisson, so that the latter can be swung well clear of the entrance. The entrance itself is splayed in order to admit of this arrangement. When in the closed position, the caisson is suspended from corbels in the masonry at each side, and the process of opening consists in floating it off these supports, by pumping air into a pneumatic chamber. The reverse operation of allowing the compressed air to escape causes the caisson to settle upon its bearings. Figs. 326 to 329 exhibit the construction of the caisson in detail.

Fig. 325.



Figs. 326, 327, 328, and 329.—Swinging Caisson at Dundee.

Traversing Caissons include all those whose motion is rectilinear. They may be subdivided into sliding, rolling, and floating caissons, according to the mode of travelling, but in each case they occupy a rectangular recess,

ELEVATION.

LONGITUDINAL SECTION

Fig. 330.—Sliding Caisson at Malta.

constructed in a side wall at right angles to the axis of the waterway, and in a direct line with the path along which they travel to close the entrance. These caissons are almost universally of the box type (hence sometimes called *box caissons*), consisting of a floor, side and end plating, and a watertight deck, the whole being divided into compartments according to the requirements of buoyancy and the mutability of design.

Sliding Caissons are provided with keels or rubbing plates on their undersides, by which they are hauled over sliding ways set in the floor of the caisson berth. This method gives rise to a certain amount of friction, which may be diminished to some extent by suitable flotation adjustment. Sliding caissons have been constructed at Malta, Portsmouth, Milford, and elsewhere. The following is a brief description of one used to close the Hamilton Graving Dock at Malta,* (see figs. 330 and 331) :—

" The rectangular sliding caisson, made of mild steel, is $40\frac{1}{4}$ feet high and $16\frac{1}{2}$ feet wide, exclusive of the keel and stem timbers, and is strengthened by two watertight decks, and bracing and framing. As the position of the entrance precludes heavy traffic passing over the caisson, the roadway deck could be placed low enough to pass under the covering of the camber, $1\frac{3}{4}$ feet below the coping, connection being made with the quay by a hinged flap. The caisson can be floated out from its normal position to the outer stop, thereby adding 38 feet to the available length of the dock. The air-chamber, 92 feet by $16\frac{1}{4}$ feet by $8\frac{1}{8}$ feet, in the middle of the caisson, is reached through two shafts. The caisson is ballasted by concrete blocks on the floor of the air-chamber, and by water in the tanks under the roadway deck at each end. Without any ballast, the caisson would float with the top of the air-chamber 2 inches above the water, but the concrete ballast more than balances the flotation, producing a normal pressure on the sliding ways of 10 to 20 tons. The water ballast is adjusted by means of a three-way stopcock in the 4-inch pipe connecting the tanks, enabling the water to be run from one tank to the other, or one or both tanks to be emptied. The caisson can be hauled in or out of the camber in five minutes, by two steel pitch chains connected with the hydraulic hauling gear, and exerting a pull of 30 tons on the two projecting arms of the caisson to which they are attached. The caisson is guided into the camber by the keels and granite rubbing pieces below, and by the fenders and

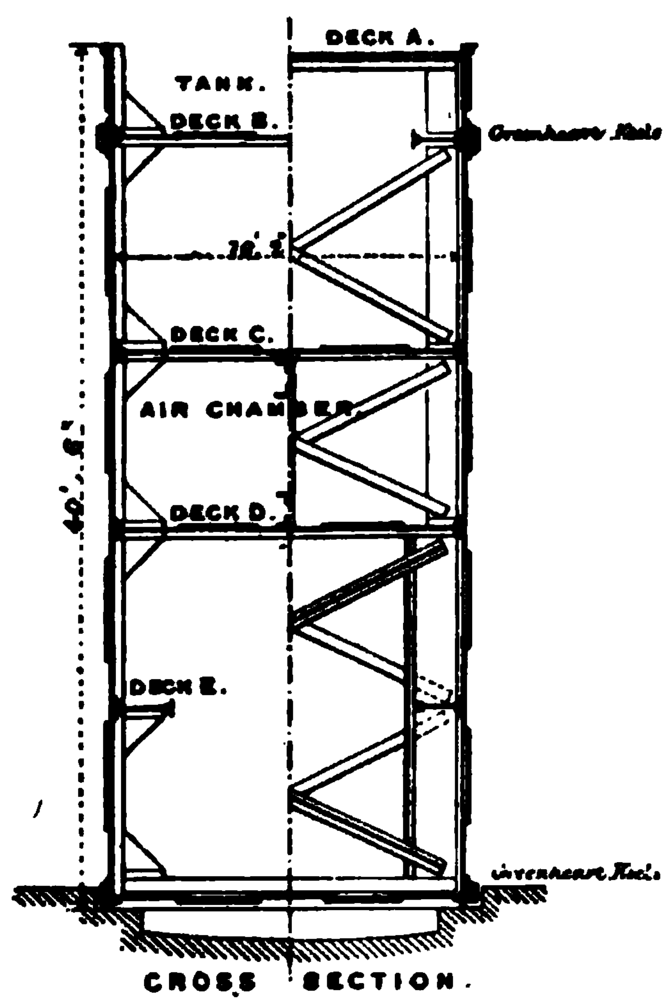


Fig. 331.

* C. and C. H. Colson on "Hamilton Graving Dock, Malta," *Min. Proc. Inst. C.E.*, vol. cxv.

rubbing pieces above, and tilting is prevented by the adjustment of the water-ballast, and by rollers on the underside of the camber girders. The caisson is stopped automatically at the end of its course into or out of the camber, and buffers are placed in the recess opposite the camber, in case of a failure of the automatic stopping gear. The maximum tensile strain on the plating of the caisson does not exceed $6\frac{1}{2}$ tons per square inch, when one side of the caisson is dry and the water is up to deck B on the other side. The keels and stems are greenheart, $10\frac{1}{2}$ by 8 inches, and the rubbing pieces and fenders are American elm. Two sluice valves, $3\frac{1}{2}$ feet in diameter, and $1\frac{1}{2}$ feet above the deck floor, furnish an auxiliary means of filling the dock. A 4-inch hand-pump serves to remove water from the air-chamber. The hauling arms can be readily moved when the caisson has to be floated out of place."

In *Rolling Caissons*, as the epithet implies, the sliding ways are replaced by rollers which are attached either to the underside of the caisson or to the pathway. This method of guidance obviates an impediment to movement, due to the slight side clearance between a caisson and its sliding ways. Often while travelling, the projecting portion of such a caisson comes under the influence of the wind, which results in its getting jambed diagonally. There is also less friction with rollers than with sliding surfaces, and, consequently, less abrasion. There is the risk, however, that the rollers themselves may get out of order, in which case any advantages they may have are more than counterbalanced by the trouble and difficulty of effecting repairs. At the same time, it is only fair to admit that, from experience of many cases, it has been found that the likelihood of such a contingency is remote.

As an example of a rolling caisson, the following description of one constructed at the sea-lock of the new Bruges Canal,* within the past few years, may be useful (see figs. 332 to 335) :—

The caisson is a steel framework, with plating of the same metal, of $14\frac{3}{4}$ feet uniform width, presenting in elevation the form of a trapezium, whose top and bottom lengths are $80\frac{1}{3}$ feet and $67\frac{1}{12}$ feet, respectively. The height of the caisson is $41\frac{3}{4}$ feet, the upper surface being 8 inches above the highest tide level. A watertight deck is laid about 16 feet above the keel, and the chamber thus formed is occupied only by the kentledge necessary for preserving equilibrium. The upper part of the caisson, though enclosed, is adapted to the free entry of water from either side, by the formation of a series of orifices, 14 inches in diameter, in each face, at the level of the watertight deck. Under the fluctuations of water level, the volume of displacement remains constant, and, consequently, the weight on the wheels remains unchanged after once being regulated by ballasting. These orifices are opened and closed, as required, by valves worked from the top deck. The caisson is carried on eight wheels, each $3\frac{1}{3}$ feet in diameter, on four axles,

* *Vide* Piens on "Portes à un seul vantail de l'écluse Maritime du nouveau Canal de Bruges," *Seventh Int. Nav. Con.*, Brussels, 1898.

arranged in pairs. The working parts are open to inspection by means of a pneumatic shaft leading to the chambers in which the axles are placed. The buoyancy chamber is also accessible by means of a similar shaft. Watertightness at the abutting surfaces of the caisson is established by greenheart facings. The caisson is designed to stand a head of water from either side. Its displacement is about 420 tons; its own weight, 196 tons; and the ballast, 273 tons; leaving some 49 tons excess weight to insure stability during movement.

The general framework of the structure comprises eight large vertical girders, placed at intervals of 8 feet, and extending to the full height and width of the caisson. The flanges of these girders serve as bearing surfaces for the plating; they are formed of 6 by $2\frac{1}{2}$ by 3 inches channel iron. The horizontal struts are similarly composed, but double.

Six tiers of horizontal joists, 14 inches deep, connect the vertical girders on each face. These joists are spaced at varying distances apart, according to the intensity of hydrostatic pressure. Between the vertical girders are three rows of intermediate bearers, only the centre one of which is prolonged above the watertight deck. These bearers are of channel iron of the same section as the vertical flanges. The watertight decking is carried by the horizontal struts of the main girders, with deck joists between and at right angles to them. The thickness of the plating varies from $\frac{1}{4}$ to $\frac{2}{5}$ inch.

Floating Caissons may either be of the box or the ship type. In the former case, they are generally rectangular in plan and similar to the examples of box caissons already described. Their distinction lies simply in the fact that they are moved entirely by flotation, without guides or rollers. Figs. 336 to 338 illustrate a floating caisson used at Blackwall, London.*

The caisson has only one meeting face, and that of teak, 14 by 7 inches. There is a lower air-chamber extending the whole length of the caisson, formed by a watertight deck at a height of 11 feet 6 inches above the bottom. Above this level, the caisson is divided into three compartments by vertical bulkheads, which are also watertight. The ballast at the bottom of the air chamber consists of cast-iron kentledge, set in Portland cement concrete. Three sluices, each 3 feet in diameter, allow water to be transmitted through the caisson, the valves being controlled by spindles passing through the air-chamber to the upper deck. The following are the sizes and general dimensions of the framing:—Angle irons at sides, 3 by 3 by $\frac{3}{8}$ and 18 inches apart; angle-iron cross beams, 4 by 4 by $\frac{1}{2}$ inches to high-water level, and 3 by 3 by $\frac{3}{8}$ inches above; centre uprights, 4 by 4 by $\frac{3}{8}$ inches, also 18 inches apart; deck beams, 3 by 3 by $\frac{3}{8}$ inches. The plating is $\frac{7}{16}$ inch thick up to the watertight deck, and above that level, $\frac{3}{8}$ and $\frac{5}{16}$ inch thick. Rock elm fenders, 10 by 10 inches, and a decking of English

* *Vide* Macalister on "Caissons for Dock Entrances," *Min. Proc. Inst. C.E.*, vol. lxv.

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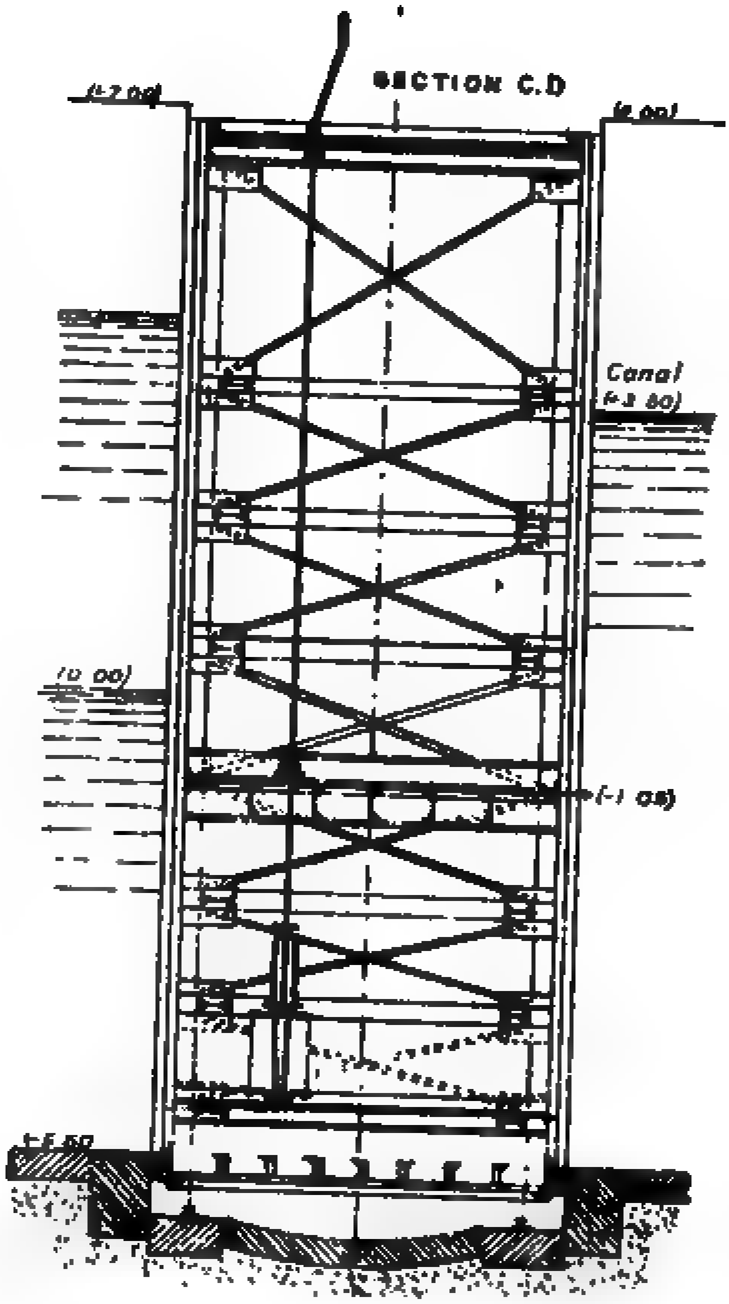

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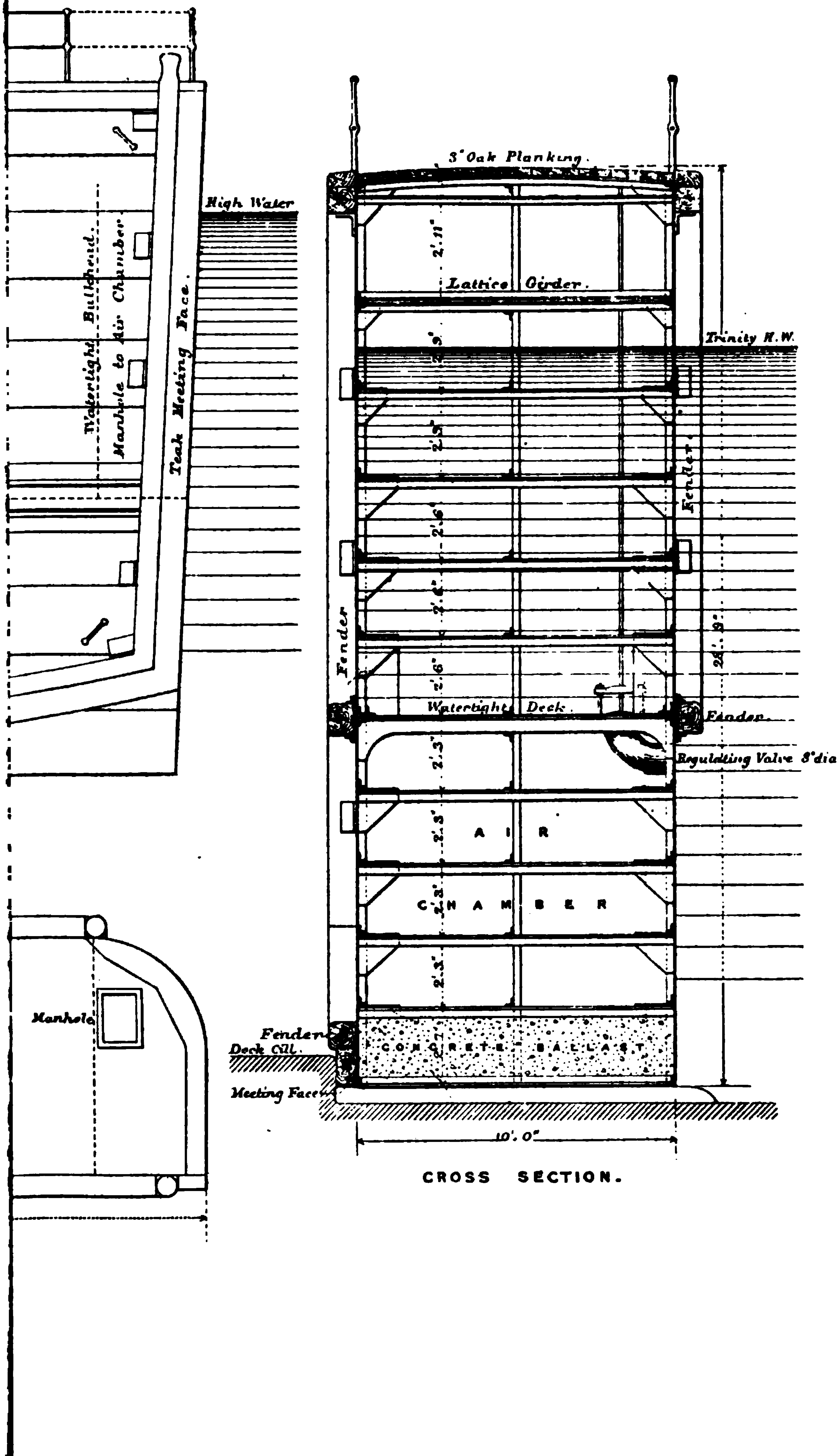

CROSS SECTION.

oak complete the general features of the caisson, which was constructed in 1878 from designs by Messrs. Kinipple and Morris.

That a caisson of this type is not necessarily rectilinear in plan is evidenced by the instance of a caisson (fig. 339), designed in 1864 by the

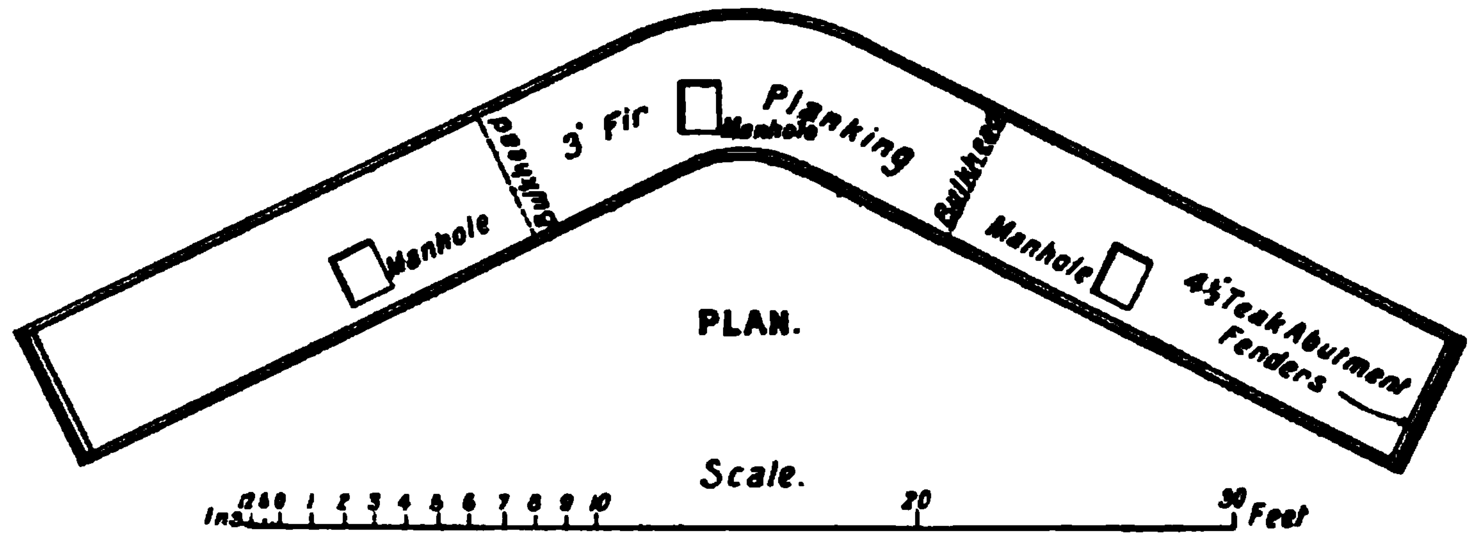

Fig. 339.

late Mr. W. R. Kinipple, and built the following year for a graving dock at Limekiln, London. Beyond the eccentricity of its form, inspired by the desire to obtain the axial advantage of a mitred sill, there is nothing very remarkable about its construction. As in the preceding case, it is furnished with a lower air-chamber and two watertight bulkheads, which latter, however, pass right through the air-chamber and completely trisect the caisson.

Ship Caissons have more or less the form of an ordinary navigable vessel, but the curvature of their sides varies very much with the depth of water in which they have to float. At the Bute Docks, Cardiff, where the draught of water on occasion is as little as 9½ feet, the caisson has had to be designed with sufficient buoyancy space at that depth to support the upper weight. This and the necessity for a margin of stability, has necessitated the somewhat peculiar profile shown in fig. 340.

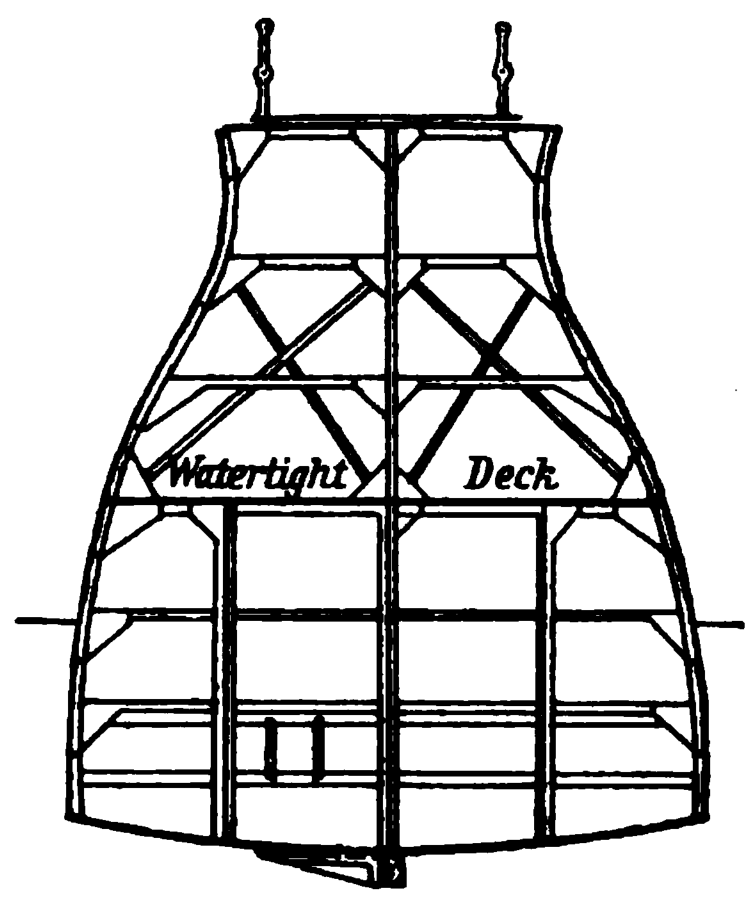

Fig. 340.—Ship Caisson at Cardiff.

The more general section of ship caisson is similar to that in fig. 341, which is the section of one at the Kidderpur Docks, Calcutta.* A plan and elevation are given in figs. 342 and 343. The keels and stems of the caisson are faced with greenheart.

A caisson differing somewhat in construction and internal arrangements is that (fig. 344) closing the entrance to the Alexandra Graving Dock at Belfast,† a short description of which is appended.

* Bruce on "The Kidderpur Dock, Calcutta," *Min. Proc. Inst. C.E.*, vol. cxxi.

† Kelly on "The Alexandra Graving Dock, Belfast," *Min. Proc. Inst. C.E.*, vol. cxi.

Figs. 341, 342, and 343.—Ship Caisson at Calcutta.

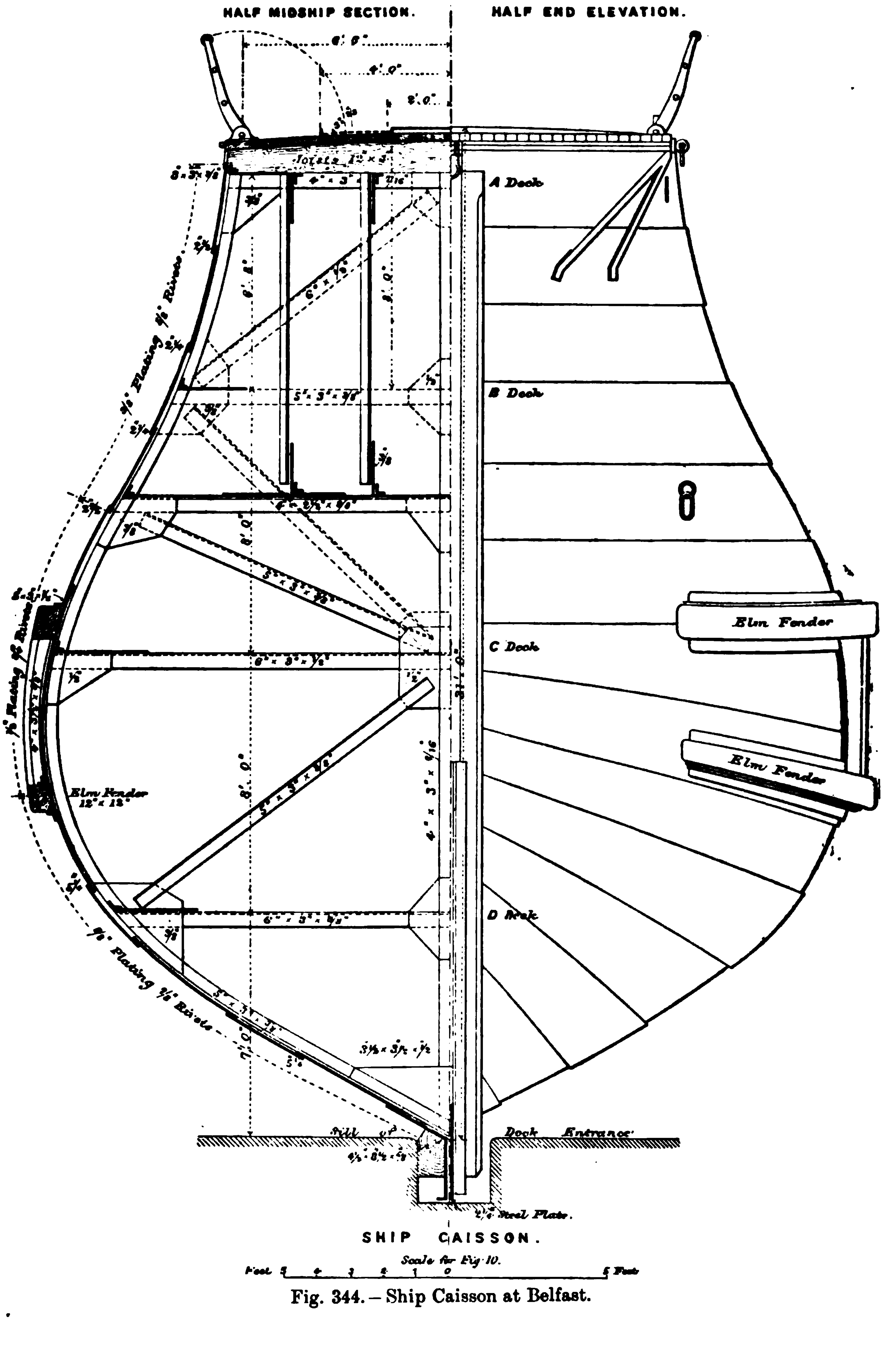

Fig. 344. – Ship Caisson at Belfast.

"The hull is of wrought iron, framed and braced together. There are four decks, the three lower of which are open lattice work, connecting the deck stringer-plates of both sides. The keel- and stem-plates are $2\frac{1}{4}$ inches thick, and the keel-base is formed of two angle-irons rivetted in the wake of the garboard strakes. On both sides of the keel and stems, heavy pieces of greenheart are bedded on and bolted to the ironwork, the timber on both sides of the keel and stems being planed true to fit the polished masonry faces of the grooves, with which they form a thoroughly watertight joint. The frames of the caisson are of angle-iron; below the level of deck, B, they are spaced at distances apart of 3 feet. The deck beams are of angle-iron. The skin plating ranges in thickness from $\frac{3}{8}$ inch at the upper part, to $\frac{5}{8}$ inch at the lower part of the caisson. The large plates are laid with their greatest lengths horizontal, in alternate inside and outside strakes, with vertical butt joints. Between decks, A and B, in the centre of the caisson, is a room with watertight floor and bulkheads, in which the engine, boiler, and pumping machinery are placed. Hatchways in the upper deck give access to the engine-room and other parts of the interior of the caisson. The roadway deck has strong angle-iron deck beams and stringers, and it is cleaded with 4-inch Dantzic oak planking, caulked and payed with marine glue. A horse-track, along the centre of the roadway, is formed of American rock elm slabs, spiked to the deck flat; and on each side of it, tracks of wrought-iron $\frac{1}{2}$-inch bars are screwed down to the decking; guides of angle-iron are fitted along their outsides; and a hinged handrail of wrought-iron gaspipe is fixed along both sides of the roadway."

The caisson illustrated in figs. 345 to 347, forming one of a number of interchangeable caissons in the service of the Mersey Dock and Harbour Board, is mainly used for work of a temporary nature during the absence of, or in case of accident to, the dock gates. It consists of four decks, below the lowest of which is located the concrete ballast. It will be noticed that the upper deck, which is of wrought-iron plating, is not available for traffic. There are three bulkheads, two transverse and one running fore and aft. These caissons do not fit into grooves, as in the previous instance, but have a single plane-bearing surface against a sill, and quoins arranged in the curved pierhead of an entrance, so that the same caisson, which is 100 feet long, can serve in several situations.

Lowering Platforms.—The difficulty of entirely recessing a traversing caisson under cover of the quay, and of, at the same time, equipping it with a suitable deck at quay level for the purposes of traffic, has been overcome by the introduction of a lowering platform. The platform, which constitutes the roadway, is supported on a series of hinged verticals, in a manner more fully described in Chapter x. A caisson designed in this manner, by the late Mr. W. R. Kinipple, closes the entrance to the Garvel Graving Dock, at Greenock.* It is a rolling caisson, with the rollers attached to

* Macalister on "Caissons for Dock Entrances," *Min. Proc. Inst. C.E.*, vol. lxv.

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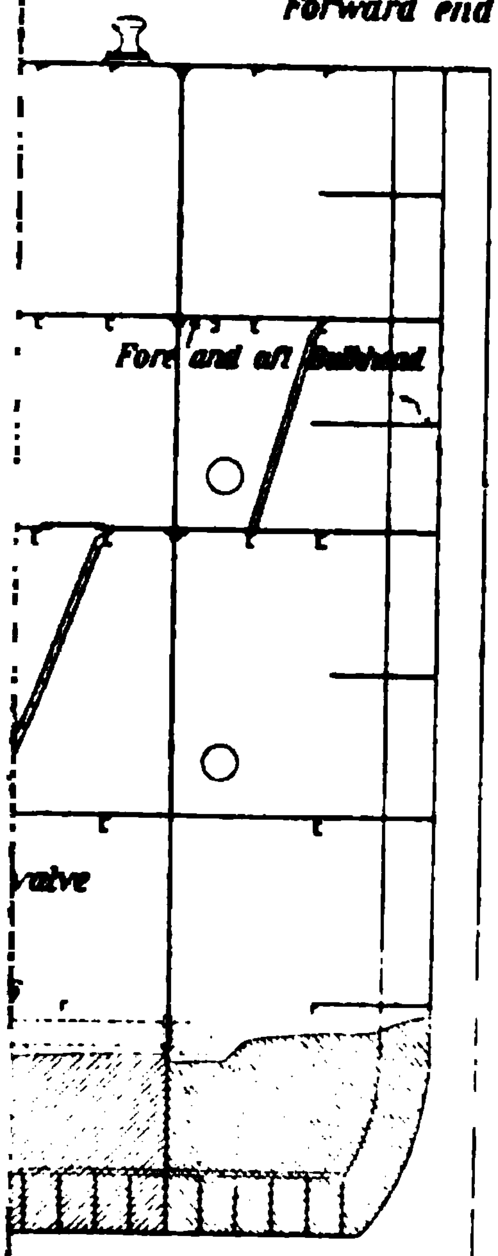
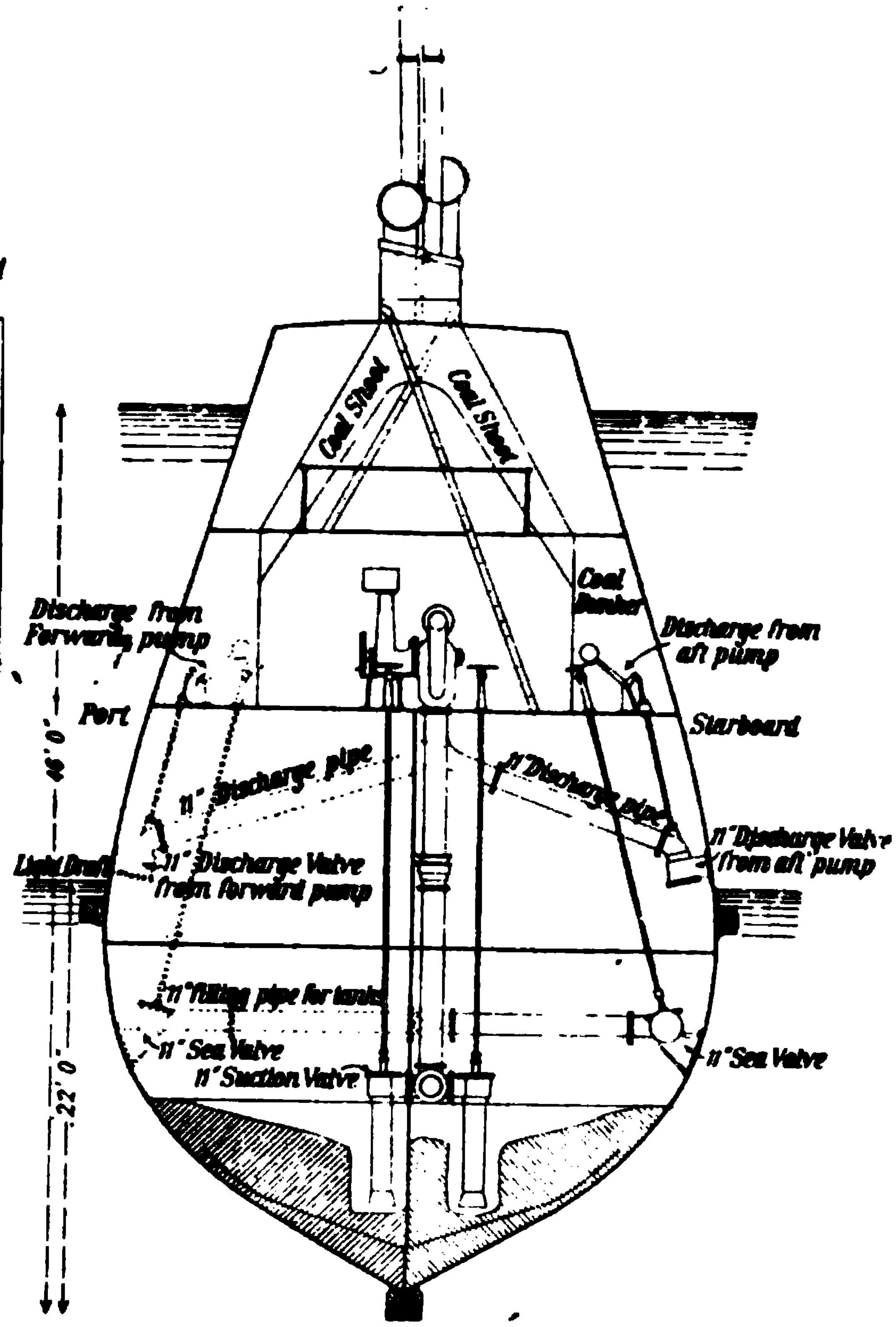
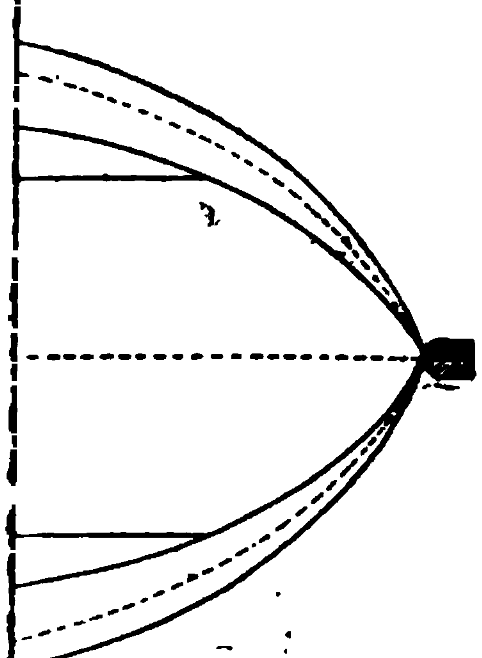




Transverse Section.



Plan

at Liverpool.

the underside of the caisson, and running upon plate rails let into the floor. A section showing the general arrangement is given in fig. 348. "The double-flanged cast-iron rollers are 18 inches diameter, and are spaced 9 feet apart. The breadth of the caisson over the greenheart meeting faces is 19 feet 10 inches, and the width between the granite faces 20 feet, giving a clearance of 2 inches. A difference of head of from 3 to 4 inches is sufficient to move the caisson from one face to the other."*

Fig. 348.—Caisson at Greenock.

Tables are appended, with statistics of size and expenditure, relating to typical caissons constructed in various parts of the world.

REFERENCES.

On the subject of stresses in dock gates, the reader who desires further information is referred to the following papers in the *Proceedings of the Institution of Civil Engineers*:—

"Strain to which Lock Gates are subjected." By P. W. Barlow. Vol. i.
"Strains on Lock Gates." By W. J. Kingsbury. Vol. xviii.
"Strength of Lock Gates." By W. R. Browne. Vol. xxxi.
"Dock Gates." By A. F. Blandy. Vol. lviii.
"Design and Construction of Dock Gates of Iron and Steel." By J. M. Moncrieff. Vol. cxvii.

Also, to a paper in the *Proceedings of the Liverpool Engineering Society*:—

"Dock Gates." By W. Brodie. Vol xviii.

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* Kinipple on "Greenock Harbour," *Min. Proc. Inst. C.E.*, vol. cxxx.

TABLE XXVI.—CAISSONS.

Port.	Entrance.	Date.	Width of Waterway.	Depth of Water on Sill at H.W.O.S.T.	Extreme Length of Caisson.	Central Width of Caisson.	Height of Caisson.	Type.	Cost per Sq. Foot of Waterway.	Total Cost.
		Circa.	Ft. ins.	Ft. ins.	Ft. ins.	Ft. ins.	Ft. ins.		s. d.	£
Halifax, N.S.,	Graving Dock,	1889	85 0 (mean)	30 0	...	...	36 6	Ship.	...	...
Biloela, N.S.W.,	,,	1889	84 0	32 0	90 0	14 9	39 2	Sliding,	110 0	17,140
Belfast,	Alexandra Graving Dock,	1889	80 0 (coping level)	31 0	...	26 6	33 0	Ship,	35 0	4,380
Barry,	Graving Dock,	1893	60 3	26 9	63 3	17 0	34 6	Floating,	47 2	3,800
Greenock,	James Watt Dock,	1886	75 0	32 0	...	20 0	41 0	Rolling.	...	...
Buenos Ayres,	Graving Dock,	1898	65 6	25 0	67 3	24 0	36 6	Ship,	87 9	7,180
Greenock,	Garvel Graving Dock,	1874	60 6	20 0	...	16 0	25 6	Rolling,	100 0	6,050
Monte Video,	Cerro Graving Dock,	1876	54 6	24 0	61 0	9 6	29 0	,,	...	...
Liverpool,	General,	1897	100 0	39 0	103 6	30 6	49 3	Ship,	45 6	8,870
Quebec,	Pont Levis Graving Dock,	1882	62 0	25 6	68 0	15 10	32 0	Rolling,	72 1	5,700
Malta,	Hamilton Graving Dock,	1893	94 0	36 6	96 0	16 6	40 6	Sliding,	85 7	14,690
Bruges,	Sea Lock,	1898	65 6	33 0	80 3	14 9	41 8	Rolling.	...	...

TABLE XXVII.

The following data relate to the Caissons closing the entrances of the principal French Graving Docks.*

Harbour.	Dock.	Length. Total.	Length. Keel.	Height.	Surface.	Weight. Without Cargo. Total.	Weight. Without Cargo. Per Sq. Foot.	Weight. With Cargo.	Cost. Total.	Cost. Per Sq. Foot.	Remarks.
		Ft. ins	Ft. ins.	Ft. ins.	Sq. feet.	Tons.	Ton.	Tons.	£	£	
Dunkirk, ...	2 and 3	65 11½	46 8½	28 10½	1625	88	·054	156	2400	1·48	Cartway.
,, ...	4	92 0½	69 8	33 9½	2724	186	·068	286	4560	1·67	,,
Calais, ...	4	72 6	72 6	36 1	2616	253	·097	278	7090	2·71	,,
Havre, ...	3	55 7	48 3	27 7	1430	113	·079	175	7090	4·94	,,
,, ...	4	102 0	95 3	36 7	3608	282	·078	676	7425	2·27	,,
,, ...	5	69 2	59 3½	36 1	2318	206	·089	422	9839	4·25	,,
,, ...	6	56 0	45 5½	33 3½	1690	147	·087	267		5·75	,,
St. Nazaire, . . .	1	85 3½	68 11	34 1½	2744	203	·074	415	9850	3·59	
,, ...	3	62 4	49 2½	34 1½	1994	151	·076	269	6619	3·30	
La Pallice, . . .	1	75 3	75 3	36 9	2765	206	·077	387	6300	2·28	
,, ...	2	49 0	49 0	36 9	1640	121	·074	208	3772	2·30	
Bordeaux, . . .	2	74 7½	74 7½	34 9	2596	109	·042	458	6105	2·35	Cartway.
Marseilles, . . .	1 and 2	82 8	44 3½	31 2	1979	111	·056	192	...	...	
,, ...	3 ,, 4	55 4	36 3½	27 10½	1277	72	·056	131	...	...	
,, ...	5 ,, 6	55 4	36 3½	27 10½	1277	72	·056	131	2600	2·04	

* H. Desprez on "Dock Installations," *Ninth Int. Nav. Cong.*, Düsseldorf, 1902.

CHAPTER IX.

TRANSIT SHEDS AND WAREHOUSES.

Extent of Accommodation Required—Proportion of Goods to Quayage—Statistics of Sample Cargoes—Accessibility of Sheds—Proximity to Edge of Quay—Level of Floor—General Diversity of Practice—Features of Construction—Doors and Doorways—Compartments—Lighting—Materials for Floors—Fire-resisting Construction—Monier, Hennebique, and Cottançin Systems—Pressure Sustained by Floors—Columns and Piers—Strength of Columns—Roof Coverings—Weight of Shed Roofs—Examples of Sheds and Warehouses at Tilbury, Liverpool, Dundee, Greenock, Glasgow, Manchester, Antwerp, Rotterdam, Havre, Marseilles, Calais, Dunkirk, Dieppe, Rouen, Bremen, Hamburg, Calcutta, and Buenos Ayres.

Few articles of commerce are altogether unaffected by exposure to climatic conditions, and for by far the greater quantity of goods deposited on dock quays, some protection from the vicissitudes of the weather is absolutely essential. This is provided, in most cases, in the form of transit sheds and warehouses. The former class are for the temporary accommodation of discharged cargoes, or of freights on the eve of shipment. The latter class are for the reception of goods which, having reached their destination, are to be stored for periods of longer, and probably indefinite, duration. In bonded sheds and warehouses, dutiable articles may remain under customs' seal until such time as the consignee has need of them, the imposts meanwhile remaining in abeyance.

Extent of Shed Accommodation.—The area of quay space allocated to storage purposes will necessarily depend upon several considerations. It is not always practicable to provide shed accommodation commensurate with the cubic capacity of vessels frequenting the berths, neither is it, in other instances, essential or advisable to do so. Under certain circumstances, goods may be removed from the quays almost, if not quite, as rapidly as they are discharged from the ship's hold. This happens when a cargo, even if not entirely homogeneous, is fairly uniform in character, and is consigned to but few individuals. When, on the other hand, goods have to be broken up and sorted into numerous lots, it becomes, even with the utmost expedition, a matter of several days before they can all be despatched to their several destinations. Accordingly, it is not unreasonable nor unusual, in such cases, to allow consignees a period of seventy-two to ninety-six hours in which to claim and remove their property.

A further complication arises from the necessity of dealing, practically simultaneously, with outgoing goods. Deposited on the site ready for the

reloading of a discharging vessel, they serve to decrease the amount of available quay space, and thus interfere with freedom and rapidity of movement. It is a good plan, where feasible, for a ship to discharge her inward freight at one berth, and then proceed to another to receive her outward consignments.

Mr. Hayter * has laid it down as his opinion that 350 or, at the most, 400 tons of goods per lineal yard of quay can be dealt with per annum. But in the case of Liverpool, of British ports at any rate, this quantity has been largely exceeded, upwards of 800 tons of goods per lineal yard of quay having passed through the double-storey sheds at that port in one year. At Marseilles, 500 tons has been stated as the limit; but, on the other hand, 1,000 tons is no unusual allowance at Russian ports, and as much as 2,000 tons per lineal yard have been accommodated on certain quays at Antwerp † and Liverpool. The ensuing table gives detailed instances of the ratio of the registered tonnage of vessels to the length and area of the berths occupied.

TABLE XXVIII.—Comparison of Number and Nett Registered Tonnage of Vessels Discharged and Loaded during One Year at certain Appropriated Berths in Liverpool Docks, with Length of Quay Space and Area of Shed Accommodation involved.

Berth.	Quay Space in Lineal Feet.	Shed Area in Sq. Yards.	Vessels Worked.		Days Occupied		Proportion of Tonnage.	
			No.	Tonnage.	Dis-charging.	Loading.	Per Lin. Ft. of Quay.	Per Sq. Yd. of Shed.
A	2000	16,727	114	428,729	309½	247	214·36	25·03
B	1408	12,594	102	310,818	315	229	220·75	24·68
C	900	7,970	58	180,548	132	153	200·5	22·65
D	800	8,048	63	141,236	188	171	176·54	17·67
E	900	9,108	57	179,402	144	158½	199·33	19·69
F	438	3,467	41	68,264	19	178	164·98	19·68
G	566	6,639	37	89,995	147½	88½	159·18	13·55
H	1400	13,187	91	278,639	267½	189	199·02	21·13
I	1103	10,088	80	210,886	152½	239	191·19	20·9
J	708	7,120	26	51,157	58½	38	72·25	7·18
K	716	7,249	43	119,279	166	153	166·57	16·45
L	703	9,637	63	159,097	152½	78½	226·31	16·5
M	200	714	70	19,384	59½	2	96·92	27·28

The sheds in every instance were single-storey sheds.

Where the nature of the traffic is variable, it is evident that no correlation whatever between its amount and the area or length of quay space is possible. A shed may be used at one time for the reception of grain in bulk, at another for cotton in bales, at another for provisions in boxes. The

* *Min. Proc. Inst. C.E.*, vol. c., p. 44.

† *Proceedings*, Seventh Inter. Nav. Cong., Brussels, 1898.

width of sheds will, accordingly, be regulated almost entirely by the land available for the purpose, and no other limit, apparently, can be suggested. From the smallest dimension consistent with practical utility, sheds have been constructed to such great widths as 150 feet at Liverpool, 190 feet at Manchester, and 196 feet at Havre. In the case of Manchester, however, it should be pointed out that the shed is traversed at its centre by a roadway, included, therefore, within the roof.

As indicative of the extremely heterogeneous character of some cargoes the following analyses of representative cases will be interesting and not inappropriate:—

List of cargo discharged in London, Sept., 1897, from s.s. "Milwaukee," 470 feet by 56 feet by 34 feet 9¾ inches*:—

514 head of cattle.	200 bags of starch.
132 horses.	189,200 bushels of corn.
640 sheep.	20,025 boxes of cheese.
18,412 bushels of oats.	399 cases of apples.
1,209 bales of hay.	11 cases of machinery.
13,149 sacks of flour.	16,737 deal ends.
51,629 pieces of deal.	5,723 pieces of birch plank.
16,328 boards.	134 radiators.
4,398 pieces of lumber.	830 pails of lard.
195 tierces of lard.	5,730 bags of grape sugar.

This is said to be the largest cargo discharged in London up to that date. In this condition the ship had 11,100 tons dead weight. It is reported she was discharged in 66 working hours.

This may be compared with the list of cargo carried by the s.s. "Oevic" on her maiden voyage in 1894:—

500 head of cattle.	10 cases of varnish.
2,330 sheep.	27 cases of axes.
9,061 bales of cotton.	33 cases of woodware.
14,778 pails, tierces, barrels, and firkins of lard.	20 barrels of metal polish.
3,006 boxes of bacon and ham.	13 cases of agricultural implements.
1,000 bundles of shooks.	120 barrels of grease and oil.
175 boxes of meats.	250 barrels of scale.
11,642 bags of copper matte.	1,800 sacks of oilcake.
6,532 pieces of oak.	2,352 pigs of lead.
885 barrels of oil.	160 boxes of cheese.
5 barrels of bladders.	1,250 sacks of flour.
3 coops of fowls.	1,000 barrels of resin.
100 barrels of glucose.	5 barrels of rope covering.
803 cases of canned meat.	5,000 bags of grape sugar.
100 tierces of beef.	4,897 oak staves.

* De Russett on "Recent Improvements in Cargo Steamers," *Eng. Conf.*, London, 1899.—Vide *Engineering*, June 16, 1899.

The following are the records of actual dead-weight cargoes discharged at Liverpool at the dates named:—

	"Georgic," July, 1899.		"Cymric," August, 1899.		"Cymric," October, 1900.	
	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.
General cargo, . .	4,517		5,084		3,504	
Bulk grain, . . .	5,118		4,665		6,193	
Fresh meat, . .	611		612		567	
	—	10,246	—	10,361	—	10,264
Live stock, . . .		696		575		687
		10,942		10,936		10,951

The diversity between weight and capacity is illustrated by the sample cargoes given below:—

Name of Ship.	Length.	Gross Registered Tonnage.	Cargo.			Area of Shed Occupied.
			Tons Weight.	Tons Measurement.	Cubic Feet.	
	Feet.					Sq. Yards.
"Horace," .	350	3,335	2,959	...	5,436	...
"Cymric," .	585½	12,647	9,749	13,390	535,600	18,647
"Georgic," .	558½	10,077	9,209	11,112	444,480	18,647
"Celtic," . .	681	20,880	6,102	15,644	625,760	18,647

One ton measurement is equivalent to 40 cubic feet of the ship's hold occupied by actual cargo. The ship's gross registered tonnage is based on her total content, calculated by certain rules and divided by 100.

The cargoes have been purposely chosen to exhibit a wide range and contrast.

Accessibility of Sheds.—Under all these mutable conditions one thing, at any rate, is perfectly clear—viz., that the means of access to a shed, and the facilities for the transference and removal of its contents are points of vital importance. It will be well then to briefly consider what steps may be taken to achieve the ideal result.

Considerable divergency of opinion will be found to exist in regard to this question at various ports, due mainly to conditions peculiarly local. For there are no less than four ways in which oversea goods may be despatched to their final destinations, and each of these obtains to a greater extent than the others at some locality and demands special measures. They are as follows:—

1. By direct transfer to coasting vessels, barges, lighters, and other river and canal craft.

2. By direct transfer to railway trucks and waggons.

3. By direct transfer to lorries and vehicles. In this case the distance the goods are to be taken will not be great.

4. By temporary discharge upon the quay and subsequent transference by canal, rail, or road, as the case may be.

These methods may be found both singly and in combination at the same port. With the first, however, we need not concern ourselves as it is outside the scope of the present section. The second and third methods may be considered conjointly as representative of *direct transfer* in contradistinction to the fourth method which we will term *indirect transfer*. It is not difficult then to understand that based upon these methods there have arisen two separate and distinct systems of transit sheds, viz.:—(1) Those in which the shed fronts are brought very close to the face of the quay wall, leaving only a narrow margin of from 5 to 10 feet for foot traffic; and (2) those in which the sheds are situated at a distance back from the edge of the quay, sufficiently great to admit of two or more lines of railway running parallel to the quay within the space intervening between the shed and the dock.

The latter type of shed is in vogue at Marseilles, Hamburg, Bremen, and most Continental ports, which may be called ports of transit. It is eminently suited to those cases in which a ship's freight is transferable without the necessity of selecting and sorting. The former system is practised at Liverpool, the older docks at London, and in other places where reverse conditions obtain and goods require subdivision before removal. Such ports may be distinguished as ports of destination. Sometimes the two classes of shed are exemplified at the same place, as at Manchester.

Of the two lines of rails at the dock side, that nearest the water will generally be used for the loading-off cranes. The second will accommodate the trucks to be loaded, and a third line may advantageously be added as a siding. Quay cranes, however, of broader gauge than the regulation 4 feet $8\frac{1}{2}$-inch track, if placed on pedestal platforms, as is frequently the case, admit of a line of trucks passing beneath and between them, thereby producing a considerable saving in quay space. The drawback to the arrangement is a lessening of the stability of the crane. Occasionally, cranes may be found located, so that the outer end of the pedestal runs upon a rail at the quay level, while the inner end is carried on a rail fixed to some part of the shed structure, as in fig. 393.

When the shed is close to the quay the discharging cranes must necessarily be situated entirely upon the shed, either at the roof or some intermediate floor level.

The two arrangements of quay sheds are illustrated in figs. 349 and 350, which are ground plans respectively of sheds at Bremen and Liverpool.

A considerable portion of a ship's cargo may be raised from the hatches by the ship's own appliances, and trucked ashore on gangways, or even, when the vessel's sides are at some height above the quay, discharged by

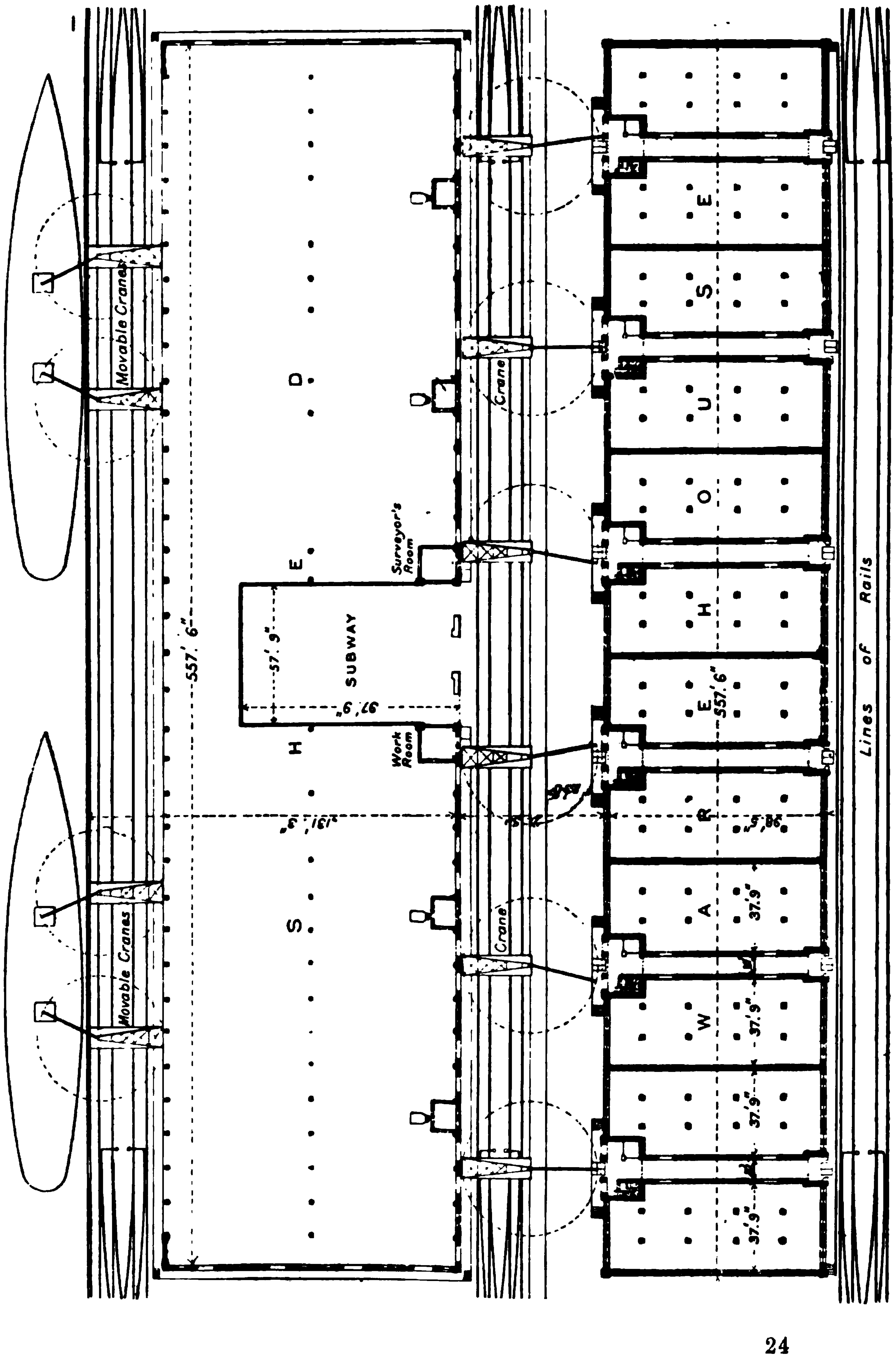


Fig. 349.—Plan of Shed and Warehouse at Bremen.

means of slides. Where there are no cranes these methods must obviously be adopted; but the question of unloading appliances is more suitable for discussion under the section of Working Equipment.

On the landward side of the shed, will generally be found a roadway for cart traffic, often in conjunction with additional lines of railway.

The level of the shed floor is another point concerning which opinion is divided. At some ports it coincides with the quay level; at others it is raised 3 feet or more above the quay, the object in the latter case being to bring it on a plane with the floors of waggons and carts so as to facilitate trucking. This method forbids, while the alternative method allows, carts and vehicles to enter the shed, and so to a certain extent to obviate trucking. Local practice, again, influences the decision as to which method in preferable.

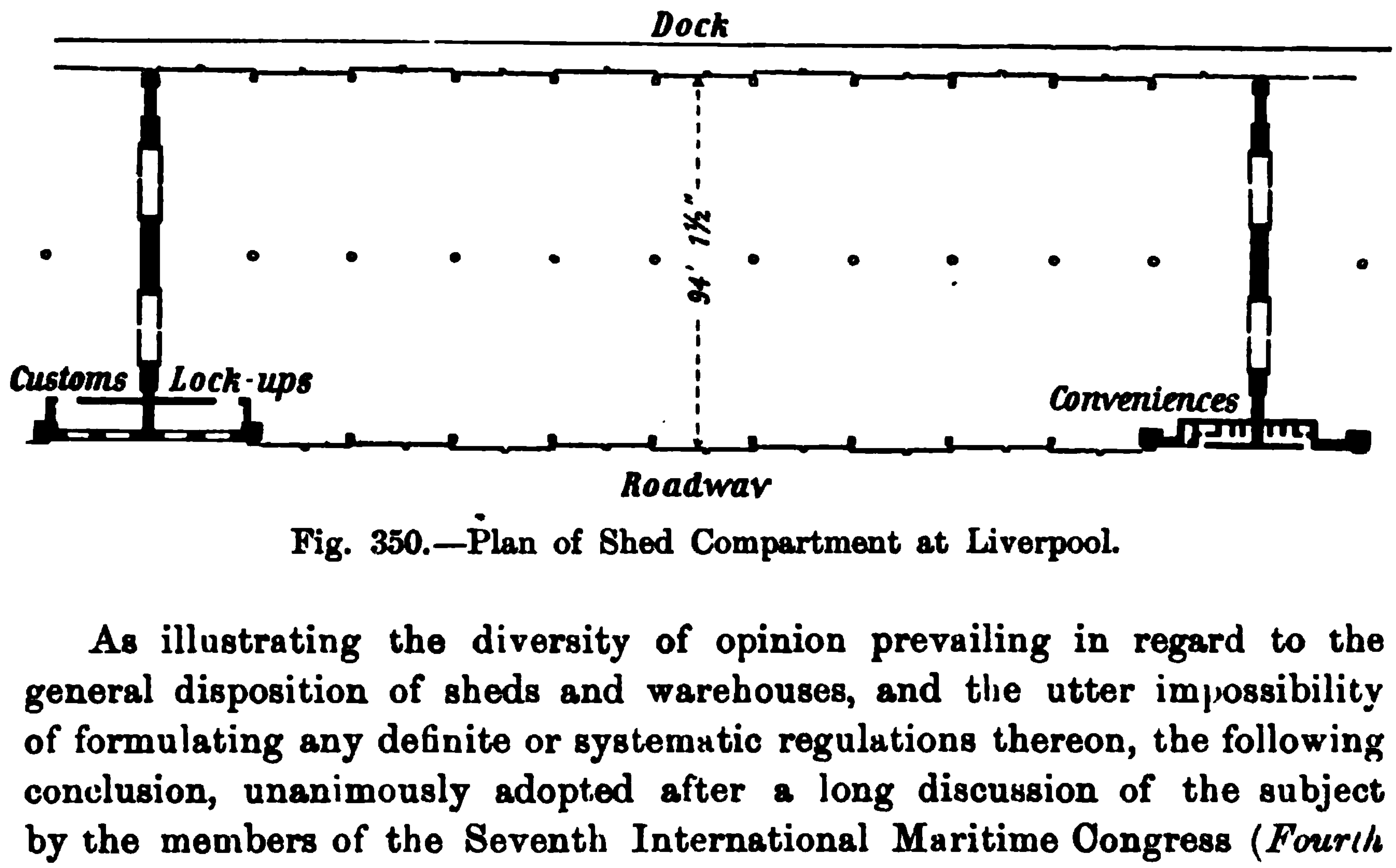


Fig. 350.—Plan of Shed Compartment at Liverpool.

As illustrating the diversity of opinion prevailing in regard to the general disposition of sheds and warehouses, and the utter impossibility of formulating any definite or systematic regulations thereon, the following conclusion, unanimously adopted after a long discussion of the subject by the members of the Seventh International Maritime Congress (*Fourth Section*—Seaports) sitting at Brussels in 1898, may be quoted:—

"*Question.*—Warehouses and sheds: accommodation, size, mode of construction, means of access.

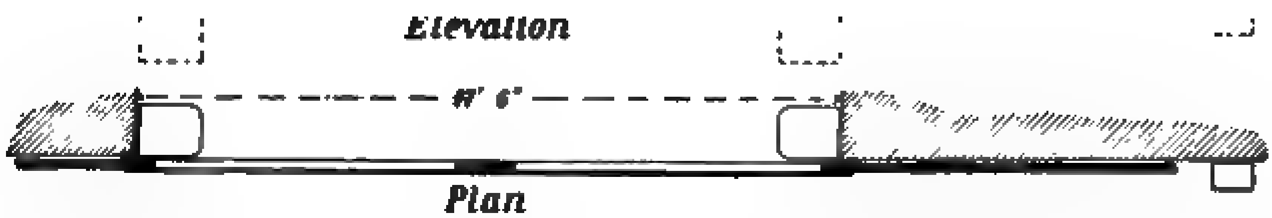
"*Conclusion.*—Considering the preponderating influence which variable elements in the different ports, especially the nature of the traffic and the commercial customs, must have on the conditions of the establishment of quays and warehouses, the Fourth Section is of opinion that there is no occasion to draw up general rules with regard to these conditions of establishment, as the arrangements adopted in each particular case are of interest solely by way of indication for analogous cases."

Features of Construction.—Methods of shed construction fall largely under those of building generally, and it is not proposed here to discuss details which are common to ordinary structures, and for which reference may be made to any suitable text-book on building construction. Those

features alone will be dealt with which are essential and prominent from the point of view of a dock engineer.

It is manifestly desirable that sheds (and warehouses) should be, as far as possible, of thoroughly fireproof construction throughout. From motives of economy, however, the former are often constructed of inflammable material, such as timber and zinc. Single-storey sheds are most noticeable in this respect.

Doorways.—The openings in the sides of a shed, both at the dock front and along the roadway, should be as numerous as possible, more particularly in the first case, so as to be adapted for receiving the discharge from several hatchways simultaneously. It is a good plan to have continuous doors, on account of the difficulty of getting several ship's hatchways to coincide with



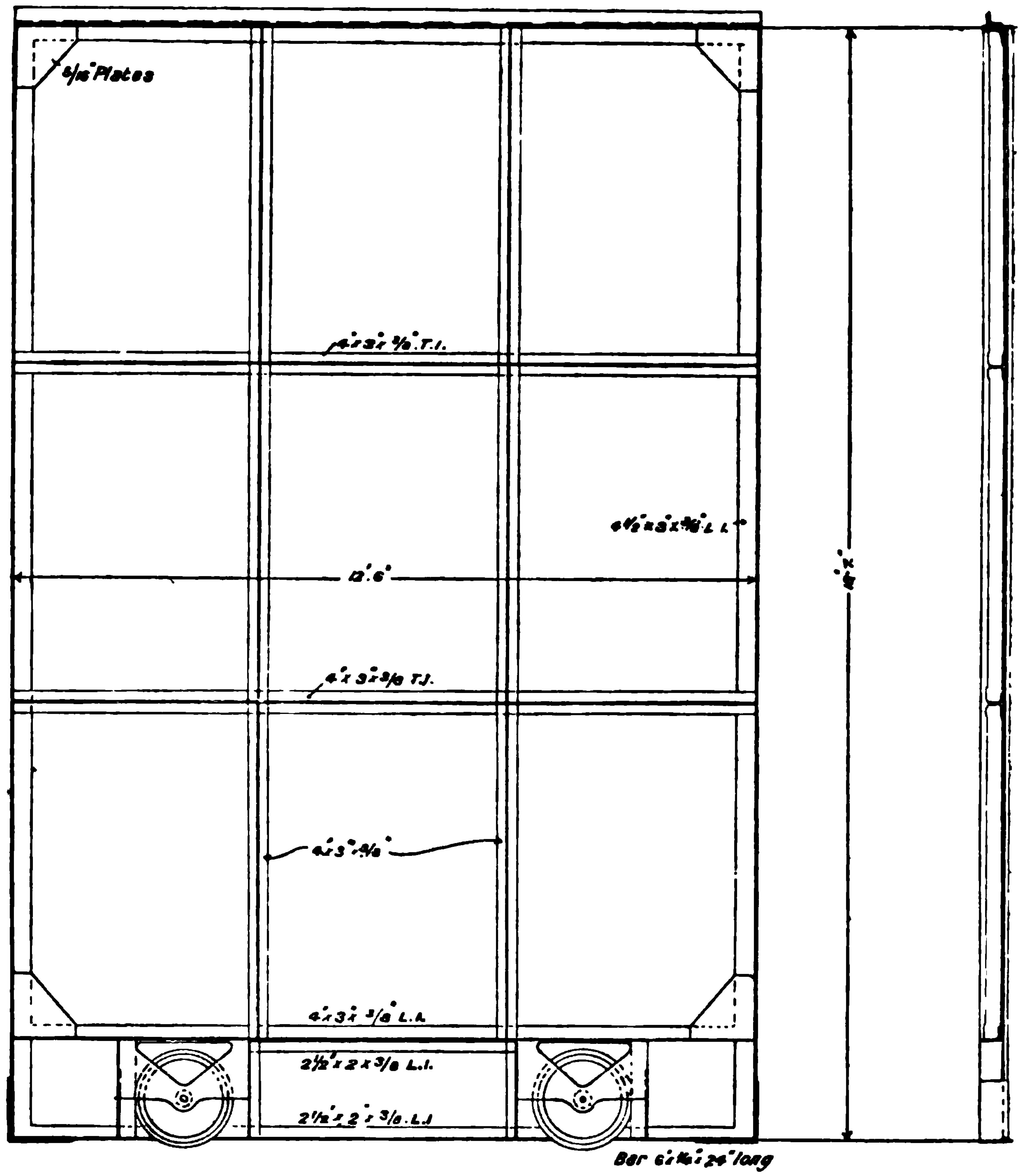
Figs. 351 and 352.—Wooden Shed Door.

isolated door openings. With this arrangement, the sides of the shed will consist of a series of columns with intervening spaces, generally closed by doors, but sometimes, as at Havre, without them. At the same time, it must be remarked that the absence of longitudinal walls causes a shed to lose much of its stiffness as a structure, and deprives it of the means of affording lateral support to its contents. Grain discharged in bulk is often prevented from spreading, on one side at least, by an external wall or partition, with a consequent saving in space, and similar assistance is rendered in the case of many other classes of goods. This fact emphasises the necessity for substantial sides to a shed. The advantages of continuous doorways, moreover, on the roadside are more imaginary than real. Not more than one-half

the entire length can be available open space, and the only benefit conferred is that of exercising some restricted choice as to its disposition.

Doors are of two varieties—rolling (or sliding) and folding.

Rolling or sliding doors consist of frames of timber or iron, with a facing of the same material. Movement is made with wheels, which run either on a ground rail or upon a rail above the door. The grooves in a ground rail are liable to become choked with dirt and grain, and need frequent cleaning.



Figs. 353 and 354.—Iron Shed Door—Elevation and Vertical Section.

They hold water, which in winter freezes and causes inconvenience. The use of lower wheels further necessitates an upper guide rail for the top of the door. Usually two rows of slide rails are provided, the doors being arranged in pairs to overlap slightly. Fastenings are made in the usual way by drop-bolts, hasps, &c. Fig. 351 is an elevation of a wooden, and fig. 353 of an iron, door constructed in this manner at Liverpool.

Folding doors are flexible sheetings of wood or metal, so contrived as to be wound round a roller at the top of the doorway. Details of one in use at Dundee are given in figs. 355 to 360. It is constructed of pitch-pine laths threaded on steel wire, and fastened to an iron drum, 12 inches in diameter. By means of balance weights and simple gearing, one man can, with ease, lift and lower the sashes.* Folding doors are lighter and take up less space than sliding doors. At the same time, sliding doors are stouter and offer a greater obstacle to the passage of fire.

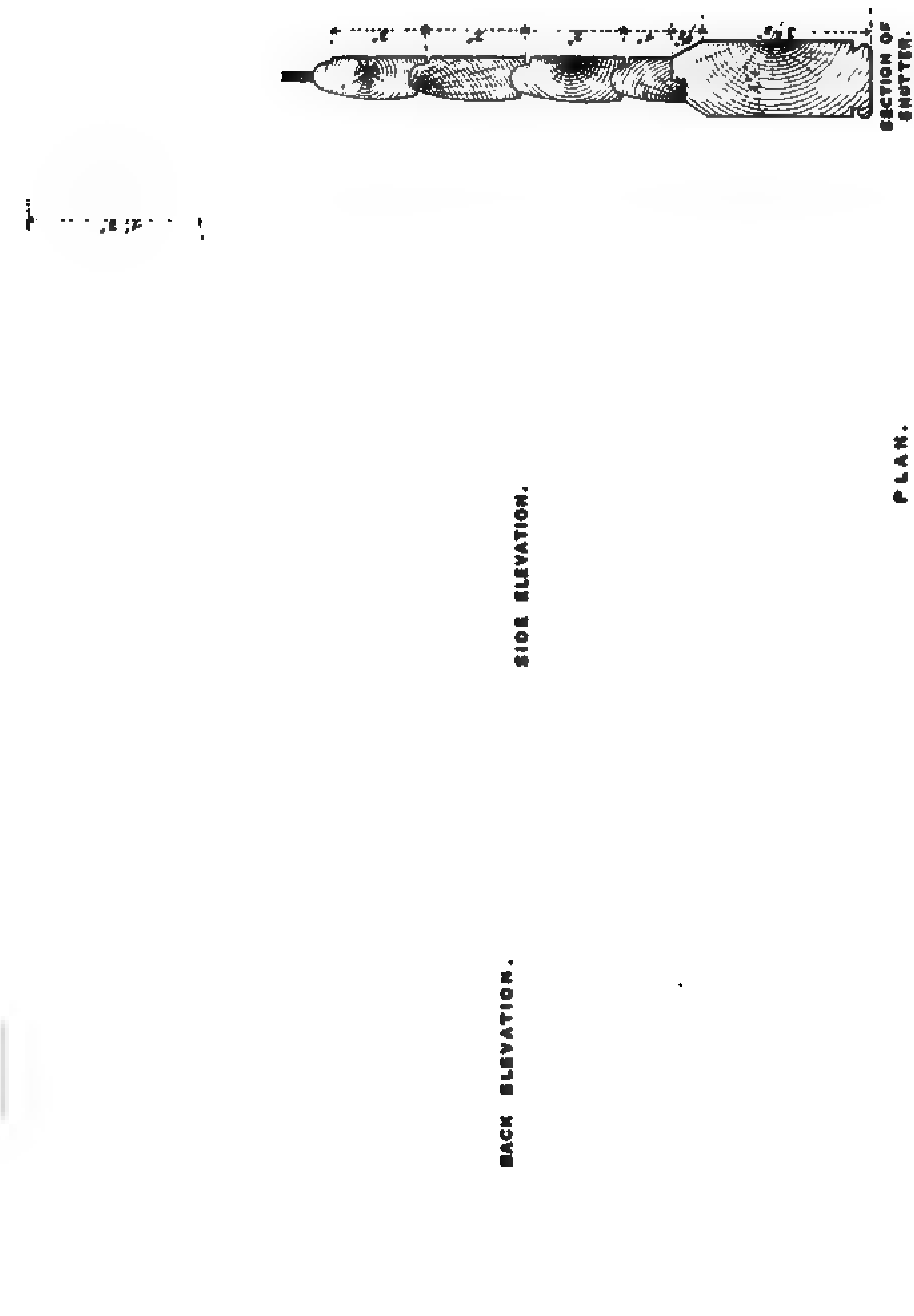
BACK ELEVATION. END ELEVATION.

Figs. 355 and 356.—Folding Door at Dundee.

The effect of fire on iron (or steel) doors is somewhat curious. Under the influence of intense heat they curl up and twist like a piece of burning paper. This erratic behaviour constitutes a source of peril, and some have even gone so far as to advocate the adoption of wooden doors on the ground that they burn away in comparative harmlessness.

Compartments.—When a shed is of considerable length, it is advisable to divide it into a series of compartments, within any one of which an outbreak of fire can be completely contined. Division walls between adjoining compartments should then be carried some 5 or 6 feet above the roof line, in order to cut off all connection. For the same reason, any door openings in such walls should be fitted with double doors. The system of detached compartments, with intervening alley ways, is a greater safeguard, but it involves less economy in space and greater expenditure in construction.

* G. C. Buchanan on "The Port of Dundee," *Min. Proc. Inst. C.E.*, vol. cxlix.



Figs. 357, 358, 359, and 360.—Details of Shutter and Mechanism.

Sheds for the reception of dutiable goods should be provided with a small office, or lock-up, in the interior for the use of the Customs'

Authorities. Public conveniences, including urinals and w.c., are useful adjuncts.

Lighting.—Single storey sheds are best lighted from the roof, either by glass tiles, skylights, or lanterns. Artificial light is also necessary for night time, and during the short days of winter. Gas may be burnt in the form of sunlights, as shown in fig. 372, suspended from the roof by chains, by means of which the frame can be lowered for cleaning purposes. Electricity is a common illuminant, and there are other systems, such as the Kitson light (burning petroleum vapour), the Lucigen light, acetylene, and others, into the relative merits of which it is unnecessary to enter here. The lower floors of sheds more than one storey in height, will necessarily derive their natural light from the sides, either through windows or glazed panels in the doors.

Shed Floors.—The nature of the material employed for the formation of shed floors is of some importance. The area may be paved, flagged, asphalted, tiled, concreted, or timbered, but it must be borne in mind that the dust arising from the wear of a stone surface is exceedingly detrimental to cargoes consisting of cereals. On the other hand, timber platforms are hardly suitable where there is vehicular traffic within the shed, and, from the point of view of fire prevention, their introduction is not to be commended. So-called asphalt floors, consisting of macadam bedded in tar, are flexible, and do not crack or fracture under concentrated moving loads, as sometimes occurs with floors of more rigid materials laid upon a yielding foundation, but their very plasticity is an objectionable feature in warm climates and in situations exposed to the direct heat of the sun's rays. Natural asphalt forms a smooth, hard, and durable surface. This and a granolithic surface, composed of equal parts of Portland cement and crushed granite, will be found to yield the least amount of dust from attrition. But the former is expensive, and the latter is only adapted to the conditions of ordinary trucking. Where vehicular traffic is heavy, a pavement of granite or whinstone setts, laid in cement on a bed of rock rubble and concrete, will generally be found the most serviceable.

Sheds of more than one storey should have upper floors of fireproof, or, at any rate, of fire-resisting material. For this purpose combinations of iron or steel and concrete are generally employed. And as this department of shed construction is of a very important character, some of the more prominent forms will be briefly noticed.

The first and earliest type was that formed of a series of iron girders connected by brick arches, the upper surface being levelled with concrete.

A later example (fig. 361) is that of a floor, formed by buckled iron plates, rivetted to the upper flanges of plate girders. A concrete covering forms a bed for Staffordshire blue tiles, $1\frac{1}{4}$ inches thick. In the instance selected for illustration the iron plates are 52 inches square.

A third form of floor, shown in fig. 362, consists of a series of rolled steel joists, 6 by 3 inches, bedded in concrete at a uniform distance apart of

26 inches. The upper surface is of granolithic concrete to a depth of 2 inches. The main joists are 12 feet apart.

The foregoing examples constitute very heavy types of floor, in proportion to their strength. With a view of minimising the amount of material, and reducing the cost of construction, various systems have been proposed in recent years, chiefly founded upon the intimate incorporation of iron or steel and concrete in one mass, and in such a way that each exercises its characteristic strength to the best advantage. One or two of the more important of these systems may advantageously be described, as there can be little doubt that the combination of these two fire-resisting materials is capable of effecting a great and useful saving in structural weight.

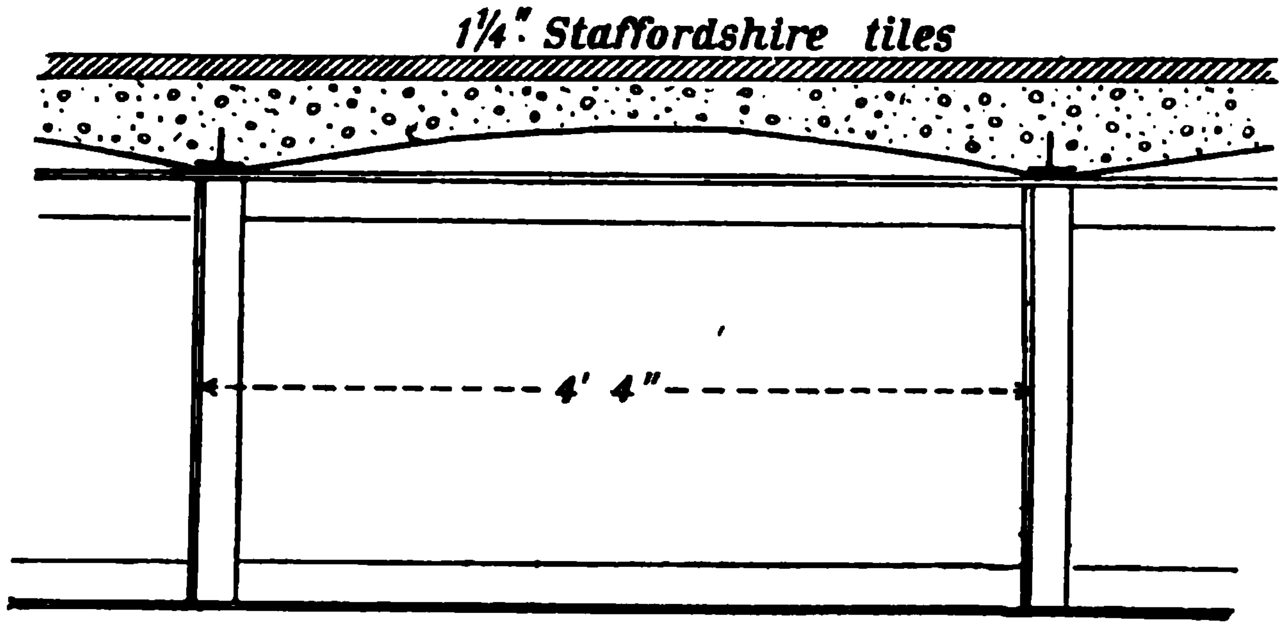


Fig. 361.—Shed Upper Floor.

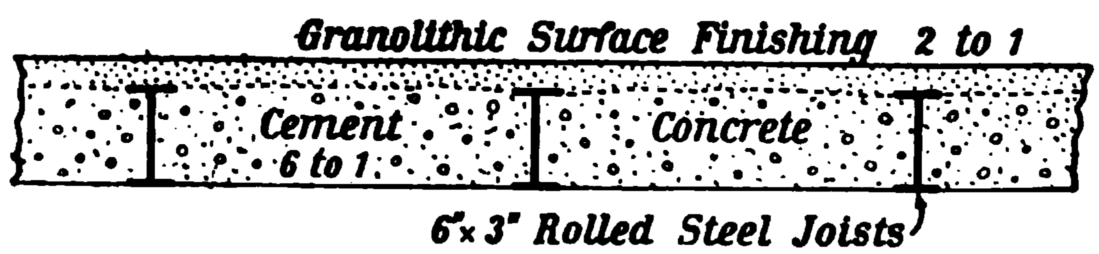


Fig. 362.—Shed Upper Floor.

Monier System.—The Monier system consists of a network of metal bedded in a concrete slab, the network being formed by two rows of bars or wires crossing one another at right angles. The lower row are the stressed bars. They are intended, in flat floors, to relieve the concrete of its tensile stress, and consequently are proportioned in number and size to the load to be carried and the amount of span. In arched floors they assist in taking up the compressive stress. The function of the upper row of bars is merely to distribute the pressure evenly, and they are generally made three-fourths of the diameter of the lower bars. The floor is divided into bays by a series of iron joists, upon which the network is laid. It is recommended that the width of the bays should not be too small. "Fairly large spans enable the supporting joists to be more economically designed, on account of a better proportion of depth to length being obtained."* At the same time, the floor must not be made unduly deep or it will prove an arrangement of dubious economy. "The minimum thickness of the concrete,

* Beer on "The Monier System of Construction," *Min. Proc. Inst. C.E.*, vol. cxxxiii.

under ordinary conditions, considered exclusively of any wearing surface, may be 1½ inches for flat floors and interior roofs and 2 inches for arched floors and exterior roofs, while 3 and 8 feet may be considered as minimum spacings for flat and arched floors respectively. Arched floors are generally constructed with a rise of only one-tenth the span; the thrust, where much weight is supported, is therefore considerable. Provision for the thrust may be supplied by tie-rods in the end bays of a floor or by horizontal end girders suitably anchored to the walls—the latter method, where possible, being preferable. Further, when a series of arches succeed one another, care should be taken that their centre lines meet on the vertical centre lines of the girders which carry them, for a very small divergence will cause an appreciable tendency to twist. This tendency may be further guarded against by embedding the girders in concrete. It is customary with ordinary flooring arches (which probably partake more of the nature of a girder than an arch) to allow a series to finish with its end member resting simply on a brick corbel; this should not be attempted with Monier arches, but a shallow, wide joist should be used as a wall-plate."*

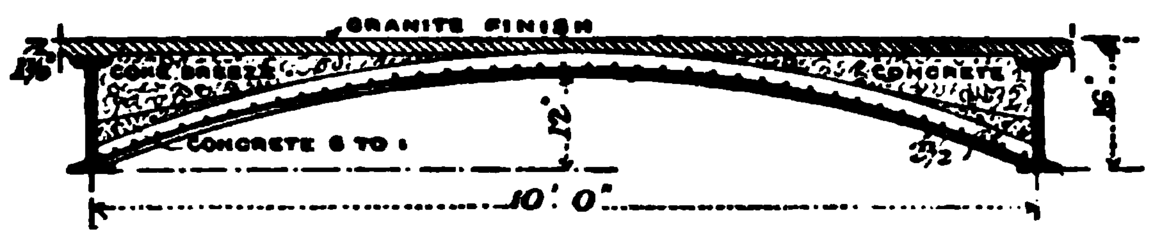


Fig. 363.—Monier Floor.

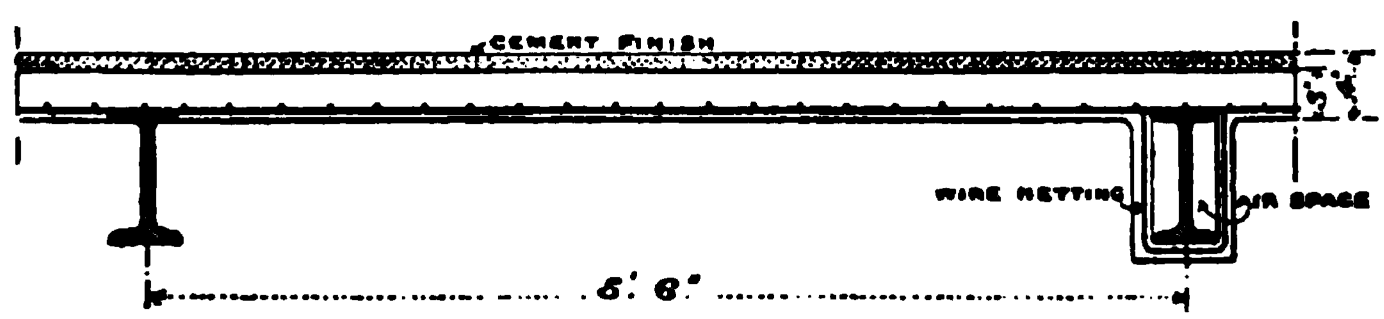


Fig. 364.—Monier Floor.

Examples of floors constructed on the Monier system are reproduced in figs. 363 and 364, from Mr. Walter Beer's paper, from which quotations have been already made, and in which the student will find a very interesting investigation of the nature and amount of the stresses set up in the various parts. These stresses, which have engaged the attention of several eminent mathematicians, are too complicated for analysis in these pages.

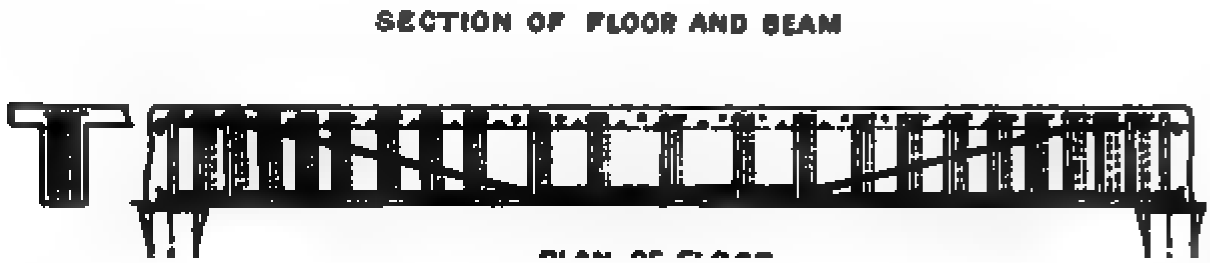
Joints are formed by causing the ends of the bars to overlap by a certain amount, which depends on the tensile strength of the bars and the coefficient of adhesion between iron and concrete, the latter being about 300 lbs. per square inch of surface. After the bars have been laid the concrete is deposited in layers, not less than 1½ inches thick, and well rammed. Thin slabs need a closer mesh than thick slabs, owing to there being greater liability to local failure.

* Beer on "The Monier System of Construction," *Min. Proc. Inst. C.E.*, vol. cxxxiii.

The lightness and slenderness of the floor call for the best materials and the most careful workmanship. The concrete should be composed of the best Portland cement, with an aggregate of broken brick or clean gravel and sand or crushed granite, in the proportion of 1 to 3. The best metal for the bars is hard steel; a soft iron does not possess a sufficiently high coefficient of elasticity. "Expanded metal," which is a network sheared out of a solid steel plate, may be used instead of disconnected bars.

Hennebique System.—This system differs from that just described more in detail than in principle. There is the same network of bars, but the meshes are larger, the bars thicker, and the parts are generally set obliquely with reference to the supporting beams. These beams are themselves constructed on the same system as the flooring.

Figs. 365 and 366 are the plan and section, respectively, of a bay of Hennebique flooring.* It will be seen that the main beam is composed of



Figs. 365 and 366.—Hennebique Floor.

three vertical rows of bars, each row containing two bars, of which the lower one is straight and the upper curved. These bars are bedded in concrete of a rectangular section, adhesion between the parts being assisted by U-shaped clips of hoop iron, which enclose the bars and extend almost to the upper surface of the beam. The model is that of a trussed beam. The concrete takes the compressive duty; the bars are simply tension rods.

The ends of the bars are either turned up or split to a fish-tail to increase the hold.

The floor illustrated has its beams 8 feet 4 inches apart, centre to centre. The latter are 8 inches wide by 14 inches deep. The floor is 5 inches thick, and was tested to a uniform load of $18\frac{3}{4}$ cwts. per square yard.

* Hope on "Construction in Fortified Concrete," *Min. Proc. L.E.S.*, vol. xxii.

The most notable application of this system is to the construction of concrete piles, which are considered in an earlier section of this work.

Cottançin's System.—The disconnected bars employed in the previous methods are replaced by a jointless wire network $\frac{1}{10}$ to $\frac{1}{8}$ inch diameter, the design varying according to circumstances. Two examples are shown in figs. 367, 368, and 369. The inventor claims for his design a large increase in strength for a given weight of metal.

There are many other proprietary systems which it would take too long to enumerate and describe. The foregoing methods are largely typical of the rest.

Figs. 367, 368, and 369.—Cottançin's Systems.

Three conditions are essential to the stability and durability of a floor compounded of concrete and metal:—

1. The metal must be completely enclosed so as to be protected from atmospheric and corrosive influences. As far as present knowledge goes, the bedding of ironwork in Portland cement mortar is attended by none of the evil results characteristic of bedding in lime mortar. Bars which have been completely embedded for lengthy periods have exhibited not the least sign of deterioration on close examination after disinterment. Exposed to the atmosphere, however, gradual corrosion is inevitable, particularly in maritime situations. Hence the necessity for a thorough covering of concrete over all parts of the metal.

2. The coefficients of expansion must be the same for the two substances, or very nearly so, within the limits of temperature to which they will be subjected. This is necessary to prevent excessive mutual stress.

3. The coefficient of direct elasticity of the metal must be greater than that of the concrete, otherwise the metal is useless. The ratio of intensity of stress in an elongated prism of the combination is $\frac{E\,A}{e\,a}$, where E is the coefficient of elasticity of the concrete, e that of the iron, and A, a, their respective sectional areas.

Shed floors should not be absolutely level. In order to get rid of any wet blown into the shed during boisterous weather, it is advisable to give the floors a rise of at least 2 inches in the first 10 feet.

TABLE XXIX., giving Cubic Feet of Space occupied by One Ton of Merchandise.

Almonds (bags),	153
Ammonia (drums),	79
Apples (barrels),	106
Arrowroot (casks),	70
Arsenic (kegs),	42
Ash poles,	62
,, lumber,	44
Asphalt (casks),	57
Bacon (boxes),	50
Bananas (crates),	132
Barley (bulk),	55
Beef, fresh (in refrigerator),	82
,, (tierces),	54
Beer (barrels),	72
,, (casks),	56
Biscuits,	245
Bran (bags),	96
Brandy (barrels),	60
Bricks, Fire- (loose),	18
Brimstone (bags),	32
Butter (boxes),	51
,, (tubs),	67
Canned fruit (casks),	52
,, meat (,,),	50
,, milk (,,),	41
Carbon (barrels),	260
Caustic (drums),	24
Cheese (boxes),	54
Cider (barrels),	56
Cigars (casks),	115
Coal,	44
,, cannel,	54
Cocoa (bags),	72
Coffee (,,),	71
Copper, ingots (casks),	13
,, pigs,	7
Corn (bulk),	50
Cotton (bales), compressed,	124
,, (,,), uncompressed,	245
,, Egyptian,	62
Currants (barrels),	51
Divi (bags),	120
Ebony (pieces),	44
Eggs,	112
Flour (barrels),	64
,, (sacks),	58
Fur skins (bales),	122
Glucose (barrels),	45
Hair (bales),	172
Hams (boxes),	50
Hares, Australian,	76
Hay (bales),	170
Hemp (bales),	95
Hides (bundles),	38
Kentledge (blocks),	6
Lard (boxes),	51
,, (firkins),	68
,, (pails),	89
Lead (pigs),	6
Leather (bales),	148
Lignum vitæ (pieces),	58
Lime acetate (bags),	110
Linseed (bulk),	50
Meal (sacks),	62
Mineral wool (bags),	266
Molasses (hogsheads),	45
Mutton, Australian,	113
Nails (kegs),	28
Nickel (barrels),	18
Nuts, Brazil (bulk),	61
,, cocoa (bags),	103
Oak (planks),	34
Oatmeal (bags),	62
Oats (bags),	66
,, (bulk),	59
Ochre (barrels),	47
Oil (barrels),	67
,, (casks),	51
,, cake (bags),	42
Oranges (boxes),	68
Ore (bags),	21
Oysters (barrels),	61
Palm oil (casks),	58
Paper (bundles),	81
,, (rolls),	74
Paraffin wax (barrels),	77
,, (casks),	59
Poultry, Australian,	62
Quicksilver (bottles),	12
Rabbits, Australian,	60
Resin (barrels),	57
Rice (bags),	47
,, (casks),	58
Rubber (cases),	51
Salt (bulk),	48
Soap (barrels),	62
Spelter (plates),	6
Sponges,	201
Starch (bags),	65
Staves (hogsheads),	78
Stearine (,,),	64
Steel blooms,	5
Sugar, grape (bags),	57
Syrup (barrels),	43
Tallow (hogsheads),	61
Tea (half-chests),	96
Tobacco (barrels),	166
,, (hogsheads),	109
,, (manufactured),	60
Walnut (logs),	28
Wheat (bags),	54
,, (bulk),	44
Wool, Australian (bales),	107
,, (without hoops),	150
Zinc oxide (barrels),	67

Weight of Animals.

Riding horse,	8 to 10 cwts.	Ox,	6 to 8 cwts.	Sheep,	¾ cwt.
Cart horse,	12 to 14 ,,	Pig,	1¼ ,,		

In calculating the strength of a floor, due regard must be paid to the weight which is likely to be placed upon it. This may be estimated from the weights of the various items of which an average cargo is composed. Table xxix. gives a series of values obtained from actual observation, but it is necessary to point out that the figures can only be regarded as approximately exact, there being frequently a considerable divergence in the extremes from which the average has been computed. It will probably be found sufficient in ordinary cases to provide for an average pressure of 3 tons to the square yard on a quay floor, and of 30 cwts. to the square yard on an upper floor, exclusive of the weight of the shed structure itself. Care should be taken to see, by official inspection, that wharfingers and others do not stack or pile goods to a height inconsistent with the weight allowed for. This is more important in the case of heavy ores, kentledge, and metal goods, which exert a vastly augmented pressure per unit volume, compared with bulkier articles.

Columns and Piers.—To avoid roofs of excessive span in single storey sheds, and upper floors of undue weight in sheds of more than one storey, intermediate supports are generally introduced in both cases. These usually take the form of metal columns or brick piers connected longitudinally by girders. Brick piers are bulky; they occupy a good deal of valuable space and obstruct light to a considerable extent. Columns, either of cast iron or steel, are better adapted to the conditions obtaining in dock sheds. Cast-iron columns are commonly circular in section and in one piece with planed bearing surfaces for the seats of the upper connecting girders. The bases may, however, be cast separately. Steel columns are usually built by rivetting together marketable forms into a rectangular or I section, bases and bearings being formed by plates with gusset stays. Hollow columns have the advantage of forming suitable ducts for rain water from the roof to the ground drain.

All columns, piers, doorway jambs, and the like should have their bases protected by metal bumpers or (granite) guard stones to a height of about 2 feet above the floor. These are designed to ward off concussions with passing vehicles. For columns, hollow castings of an approximately ellipsoidal or spherical form, bolted together in two segments and filled with concrete, will be found most suitable. Occasionally, wisps of straw have been wound round the column prior to the insertion of the concrete, in order to still further diminish the shock, but the precaution is of dubious value.

On account of the unsatisfactory behaviour of ironwork under the heat of a conflagration, columns of concrete strengthened by a hearting of metal have been proposed as a substitute for the ordinary type of iron and steel columns. It will certainly be found expedient to leave no metal surface exposed, and one valuable safeguard is to encase metal columns with external fireclay cylinders. These may be obtained in lengths of 2 feet or less; they are generally about 1 inch thick and exceed the diameter

of the column by 2 inches, the 1 inch annular space being grouted with cement. Another proposed expedient is to adapt the interior of the columns to the circulation and distribution of water.

Strength of Columns.—The strength of long columns or struts constitutes a very intricate problem. Such columns, though nominally in compression, rarely fail from a simple compressive stress; they succumb, when loaded to collapse, to a bending stress induced by the unsymmetric application of the load. Theoretically, the load should be applied so that the stress passes absolutely through the centre of every transverse section of the column, and, in that case, there would be no tendency to bind; but this ideal condition is unattainable in practice.

Let us briefly investigate the case of a column deflected by a load, W, acting at a distance, x, from the axis of symmetry. If δ be the amount of deflection produced, the bending moment at the foot will be W $(x + \delta)$. Assuming for the moment that the deflection curve is circular, as in the case of a beam of uniform strength, we have by Euclid III., 35,

$$\delta \times 2\,\mathrm{R} = l^2, \qquad (70)$$

where l is the length of the column and R the (very considerable) radius of curvature.

Now, the moment of resistance,

$$\mathrm{M} = \frac{\mathrm{E\,I}}{\mathrm{R}} = 2\,\delta\,\frac{\mathrm{E\,I}}{l^2}. \qquad (71)$$

Equating the two moments:—

$$\mathrm{W}\,(x + \delta) = 2\,\delta\,\frac{\mathrm{E\,I}}{l^2},$$

whence

$$\mathrm{W} = \frac{2\,\delta}{x + \delta} \cdot \frac{\mathrm{E\,I}}{l^2}, \qquad (72)$$

and

$$\delta = \frac{x}{\dfrac{2\,\mathrm{E\,I}}{\mathrm{W}\,l^2} - 1}. \qquad (73)$$

From a consideration of (73), if $x = 0$, δ will also be zero, unless

$$\mathrm{W} = \frac{2\,\mathrm{E\,I}}{l^2}, \qquad (74)$$

and this is the critical value for W, when the column is in a state of neutral equilibrium; so that the least increase in the load will cause indefinite bending and consequent fracture.

If the column be of uniform transverse section, instead of uniform strength, as assumed above, the curve will be one of sines, and the equation becomes

$$\mathrm{W} = \frac{\pi^2}{4} \cdot \frac{\mathrm{E\,I}}{l^2}. \qquad (75)$$

The equivalent length of a column fixed at both ends is one-fourth of that described above, and if we substitute for I, its value $\frac{\pi r^4}{4}$, and reduce to unit area, we obtain the following ultimate strength per square inch for a column of circular section, with radius r:

$$p = \mathrm{E}\pi^2 \frac{r^2}{l^2}. \qquad (76)$$

This formula (75), having originated with Euler, is known by his name. Its efficacy depends on three conditions, two of which, at least, cannot be guaranteed in practice, viz.:—

(α) The uniformity of the modulus of elasticity (E) for all fibres throughout the section;

(β) The absence of any initial deflection;

(γ) The axial position of the load.

Furthermore, it will be noticed that no allowance is made in the equation for the possible failure of the material by direct crushing, so that for short columns the calculated strength is greatly in excess of the compressive limit.

Professor Claxton Fidler shows* that if the opposite sides of a pillar whose moduli of elasticity are E_1 and E_2 respectively, be subjected to the same amount of compressive stress, one side will be shortened more than the other in the proportion of $\frac{1}{E_1}$ to $\frac{1}{E_2}$, and, consequently, of the total deflection produced by a given load, one portion causes no difference of stress and, therefore, no moment of resistance, while the remainder alone is the measure of the real moment of resistance.

The total deflection is found to be

$$\delta = \frac{\pi r}{2} \cdot \frac{e_1 - e_2}{e_1 + e_2} \cdot \frac{p}{\rho - p}, \qquad (77)$$

where e_1 and e_2 represent $\frac{1}{E_1}$ and $\frac{1}{E_2}$ respectively; $\rho \left(= \pi^2 \mathrm{E} \frac{r^2}{l^2}\right)$ is the resilient force of the ideal column in pounds per square inch; r, in this case, is the radius of gyration, and p is the actual load-intensity.

The bending moment $\mathrm{M} = \mathrm{P}\,\delta$, and the extreme stress in the fibres at a distance, y, from the neutral axis due thereto, is

$$\pm f_1 = \frac{\mathrm{P}\,\delta\,y}{\mathrm{I}} = \frac{p\,\delta\,y}{r^2}. \qquad (78)$$

Inserting the equivalent for δ from (77) and giving to $\frac{\pi}{2} \cdot \frac{y}{r} \cdot \frac{e_1 - e_2}{e_1 + e_2}$ its approximate value ·4, the maximum compressive stress on the concave side of the column,

$$f = p + f = p\left(1 + \frac{\cdot 4 p}{\rho - p}\right), \qquad (79)$$

* "Bridge Construction," chap. x.; *vide* also *Min. Proc. Inst. C.E.*, vol. lxxxvi.

which involves a quadratic in p. This formula expresses the relationship existing between the apparent stress, p, due to the load and the maximum stress, f, on the concave side. If, then, we insert the ultimate compressive stress of the material in place of f, and solve the equation, we find the breaking stress, p, of the column.

It has been assumed that the ends of the column are free to move. In dealing with a column in which both ends are fixed, the length of an equivalent round-ended column may be taken at three-fifths of the actual length.

The ultimate compressive stress in various materials may be taken as follows:—

Timber, ...	2 to 4 tons per square inch.
Wrought iron, . .	16 tons per square inch.
Mild steel, . . .	30 ,, ,,
Cast iron, . . .	40 ,, ,,

A formula very commonly used for the determination of the compressive strength of long struts, is that devised by Professor Lewis Gordon, which, using the same rotation as before, may be expressed thus—

$$p = \frac{f}{1 + a\frac{l^2}{d^2}} \quad . \quad . \quad . \quad . \quad . \quad . \qquad (80)$$

The fraction $\frac{l}{d}$ expresses the ratio of the length of the column to its diameter, or its least dimension in cross-section. The values of a are given in the annexed table.

Results obtained by this formula agree fairly closely with those given by Prof. Fidler's method.

TABLE XXX.

Material.	Cross Section.	Values of a.		
		Both Ends Rounded.	Both Ends Fixed.	One End Rounded, One Fixed.
Timber, . .	Rectangular or circular, .	$\frac{4}{250}$	$\frac{1}{250}$	$\frac{1}{100}$
Wrought iron, ,,	Rectangular, . . . Circular (solid or hollow), .	$\frac{4}{2500}$	$\frac{1}{2500}$	$\frac{1}{1000}$
,,	∟ T + □ ⌶ I ⊔ . .	$\frac{4}{900}$	$\frac{1}{900}$	$\frac{1}{360}$
Cast iron, .	Circular (solid), . . .	$\frac{1}{100}$	$\frac{1}{400}$	$\frac{1}{160}$
,, .	,, (hollow), . .	$\frac{1}{200}$	$\frac{1}{800}$	$\frac{1}{320}$
,, .	Rectangular, . . .	$\frac{3}{400}$	$\frac{3}{1600}$	$\frac{3}{640}$
,, .	Cross-shaped, . . .	$\frac{3}{200}$	$\frac{3}{800}$	$\frac{3}{320}$
Mild steel, .	Circular (solid), . . .	$\frac{4}{1400}$	$\frac{1}{1400}$	$\frac{1}{560}$
,, .	Rectangular (solid), . .	$\frac{1}{620}$	$\frac{1}{2480}$	$\frac{1}{990}$

As an example, take the case of a solid, cylindrical, cast-iron column, 12 inches diameter and 20 feet long, fixed at both ends. Then, by the foregoing formula—

$$p = \frac{40}{1 + \frac{1}{400} \times \left(\frac{20}{1}\right)^2} = 20 \text{ tons per sq. in.}$$

Fidler's formula gives 19·4 tons under the same conditions.

Roof Coverings.—The roof coverings usually employed for sheds are slate, lead, zinc, galvanised iron, felt, and roofing paper. The last-named material is inferior to the others, and should only be used for temporary and unimportant purposes.

Slate is the best roofing material, being unalterable in nature and exempt from decay. It has the drawback of being heavy, but this disadvantage is more than compensated for by its durable qualities. Large sized slates form the best kind for use, as with fewer joints there is less opportunity for leakage, and with greater weight there is less chance of the slates being lifted by the wind. For the latter reason slates should be centre-nailed, and in very exposed situations they may be additionally secured by lead or copper tingles.

Lead is a durable roof covering, but both heavy and expensive. Moreover, it is not a suitable material for steep-pitched roofs (though, perhaps, this drawback is of little importance in the case of sheds, where the roofs are generally low-pitched), owing to its tendency to creep under the influence of expansion and gravitation.

Zinc has the advantage of lightness combined with economy, but it is very subject to corrosion and decay, and is highly inflammable at a red heat. Contact with iron, copper, or lead, in the presence of moisture, produces destructive voltaic action. Lime is another deteriorating agent, as also is oak, owing to an acid which it contains.

From an exhaustive examination of a great number of zinc-covered shed roofs at Liverpool, the following valuable observations were deduced :—

1. That when zinc is in free contact with the (sea) atmosphere, a slow and gradual wasting away of the zinc takes place. The metal throws off a fine flour-like substance, which forms a deposit on its surface and is washed, or blown, away or cemented by sooty matter, as the case may be.

2. That in exposed situations the wasting away is intensified, and the surface of the zinc soon presents a roughened appearance due to close and minute pitting. Especially does this occur at the more prominent points, such as step flashings, at weather faces, at ridges and rolls, and at cappings over joints.

3. That wherever a leak occurs, and, to a greater degree, where moisture, in passing down the underside or covered upper surface of a sheet, is checked and forms into beads, as is frequently the case at the top edge of laps and joints, or where water is driven by the wind between the overlapping portions of sheets, the efflorescence lying there becomes encrusted and gradually hardens, biting into the zinc, and, in course of time, perforating it.

4. That where water lodges to the exclusion of air, as frequently occurs at the back of ridge laps or the lower end of bottom sheets, or under skylight aprons, the white efflorescence is transformed into a light umber paste which attacks the zinc in annular forms, and speedily rots it.

5. That where steam power is largely used under a roof, and the discharge from the furnace chimney is delivered within the shed, the zinc covering in the immediate neighbourhood, and especially at the openings through which the smoke makes its exit, shows signs of wear corresponding to the effect produced by a fine sand blast. The zinc also becomes soft and loses its elasticity.

Galvanised and Corrugated Iron Sheets are open to all the objections enumerated with regard to zinc, with the additional drawback that, as soon as the thin coating of zinc is perforated, the galvanic action set up between the two metals enormously increases the rate of decay. This is particularly in evidence at bolt-holes and fastenings, where the leather washers usually employed afford but imperfect protection from the access of moisture. In many instances, sheds cônstructed with galvanised sheets have had to be coated with tar or black varnish to preserve them from further ravages.

Weight of Shed Roofs.—In order to afford a rough estimate of the weight to be carried by columns and side walls, the following approximate data will be found useful. The weights are given in lbs. per square foot of horizontal area, covered by roofs of from 20 to 60 feet span:—

TABLE XXXI.

Timber trusses, including purlins,	$3\frac{1}{4}$ to $7\frac{1}{2}$
Iron ,, ,, ,,	$5\frac{1}{2}$,, $8\frac{1}{2}$
Common rafters,	$3\frac{1}{2}$
Battens,	2
Boards, $\frac{3}{4}$ to $1\frac{1}{4}$ inches thick,	$2\frac{3}{4}$ to $4\frac{3}{4}$
Slates,	5 ,, 14
Tiles,	13 ,, 20
Lead covering,	6 ,, 9
Zinc ,,	$1\frac{3}{4}$,, 2
Corrugated iron covering,	$2\frac{1}{4}$,, $3\frac{3}{4}$
Allowance for snow,	5
,, ,, wind,	20 ,, 40

From a brief but detailed consideration of some of the more salient features of shed construction, we now pass to a succinct review of the types to be met with at various ports.

Tilbury Dock Sheds, London.*

"The twenty-four berths in the branch docks are each provided with a quay shed 301 feet long and 120 feet wide, giving a floor area of over $\frac{3}{4}$ acre, the roofs being constructed with timber storey-posts, iron roof-

* Scott on "Construction of Tilbury Docks," *Min. Proc. Inst. C.E.*, vol. cxx.

principals, and boarded and slated roofs. The front and the back of each shed, for 240 feet of its length, are entirely open, but can be closed at will by steel self-coiling revolving shutters, working between the storey-posts supporting the roof. When these shutters are open, free access is afforded between the quay and the railway in the rear of the sheds, and when they are closed, the requirements of the custom-house for the safe custody of bonded goods are complied with. The ends of each shed, and the small portions of the front and back not closed by the shutters, are covered with corrugated iron supported upon timber framing. Well distributed light for the interior is obtained through 480 large glass slates in the roof of each shed. The floors of the sheds are of pitchpine planking, laid upon sleepers bedded upon a layer of ballast 12 inches thick. Quay sheds, generally similar to those for the branch docks, but of one 60-foot span, are provided for the berths in the tidal basin."

Liverpool Sheds.

At Liverpool the sheds are continuous, and their length practically coincides with the length of the quays upon which they stand. They are, however, for working purposes divided up into compartments, of which the average length in the more modern examples is rather less than 300 feet. In width they vary considerably, but the roof spans range generally from 30 to 80 feet, with a few extreme cases approaching 100 feet.

Fig. 370 shows a section of a single-storey shed, 150 feet wide, roofed in two spans. The walls are of brickwork, with doorways 20 feet wide by 16 feet and 17 feet 6 inches high. The roof trusses are a combination of wood and iron, the compression members being of wood and the tension members of iron. The intermediate supporting columns are of cast iron, and the roof covering of Vieille-Montagne zinc. The floor is asphalted.

Fig. 371 is a section of a double-storey shed, 95 feet wide, roofed in three spans. The upper floor is supported on brick piers, 3 feet square and 26 feet apart longitudinally. It is formed by main and subsidiary girders, the enclosed spaces being covered by buckled plates, upon which is laid a bed of concrete to form a level surface for a layer of 1¼-inch blue Staffordshire tiles. The roof trusses are entirely constructed in angle- and bar-iron, with riveted joints. The roof covering is Velinheli slates nailed on boarding. Continuous skylights run along each side of the ridge. The lower floor is lighted by windows in the walls and by glazed panels in the upper portion of the sheet-iron doors. In later examples of this type of shed, the width has been divided into two equal spans by means of a central row of cast-iron columns. Upon made ground the column bases are supported by concrete beds, 7 feet square, surrounding and covering the heads of two pitchpine piles, 14 inches square and about 38 feet long, driven to a firm substratum of boulder clay. The fronts of the shed, both to the quay and the roadway, consist of a series of doors closing openings,

26 feet wide, between steel columns, except at the roadside of the upper floor, where bays of brickwork alternate with doorways. The ground floor is paved with 4-inch granite cubes, bedded in gravel on an 8-inch founda-

SECTION OF SINGLE STOREY SHED 150 FEET WIDE.

Fig. 370.—Shed at Liverpool.

CRANES, MOVEABLE ALONG THE ROOF.

SECTION OF DOUBLE STOREY SHED 95 FEET WIDE.

Fig. 371.—Shed at Liverpool.

tion of concrete, well packed with rubble. The upper floor has main girders of 47 feet span, spaced 32 feet apart, and longitudinal girders connecting these at about 12 feet intervals. Upon this framework, and bolted

to it, lie rows of rolled joists, 6 by 3 inches by 16 lbs., spaced 26 inches centre to centre, forming a core for a body of concrete 8 inches in depth, which covers the top of the joists by 2 inches. The surface coating is of crushed granite passed through a sieve of 16 meshes, and retained by a sieve of 64 meshes, to the square inch, mixed with an equal quantity of cement. For the bulk of the concrete, 6 parts of broken brick and gravel to 1 of cement are employed. The ironwork throughout is of mild steel. Three-storey sheds are now being constructed on identical lines.

Fig. 372 is a cross-section of a single-storey shed of a less permanent and more economical type. The sides and end walls are of timber framing, covered with 1¼-inch Norway planking. The main uprights are of pitch-

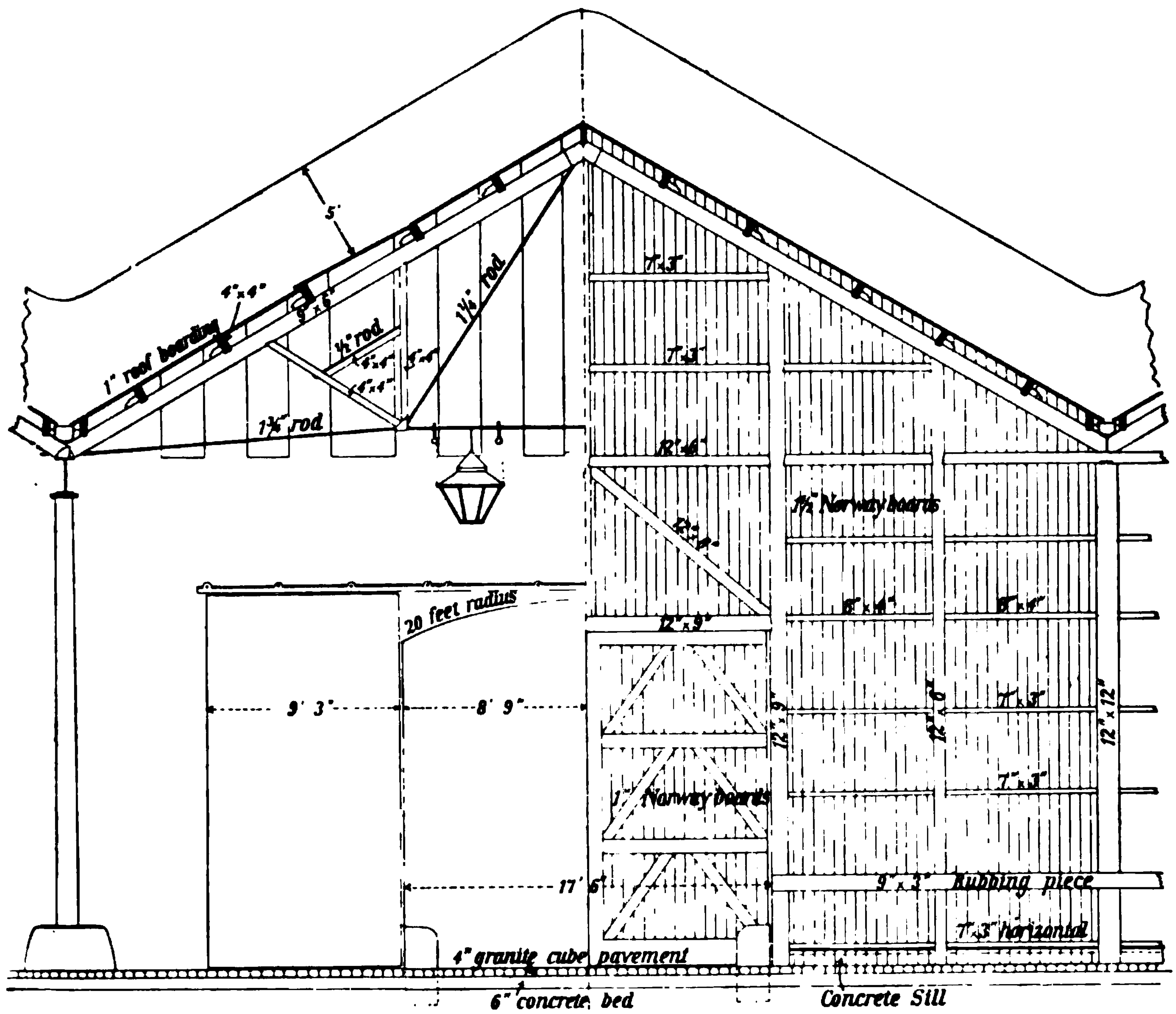


Fig. 372.—Shed at Liverpool.

pine, 12 inches square, 13 feet apart, with intermediates 12 by 6 inches, all having their bases bedded in concrete. The division walls alone are of brickwork, and this with the design of curtailing the ravages of a possible fire. The roof has combined timber and iron trusses, covered with boarding whereon is laid Graves' Patent Roofing No. 2, the laps and joints of which are coated with mastic before being nailed to the boarding.

A coat of warm mastic has then been laid over the whole of the roof surface, and covered immediately with warm, sharp sand.

Sheds at Dundee.*

"Around the docks and river quays there are single-storey transit-sheds covering an area of 45,000 square yards. A cross-section of one recently erected is shown in fig. 373. It is 300 feet in length by 120 feet in breadth, in two roof spans of 60 feet, and the height from ground level to the eaves is 13 feet 9 inches. The walls are of brick, with ashlar quoins and tabling, and there is a row of cast-iron columns along the centre of the shed supporting the roof, and a similar row on the river front, which is closed in with wooden sliding doors. The roof covering is of slate, and the principals and girders are of mild steel. The shed is floored with granolithic pavement, consisting of a 4-inch layer of broken stone, upon which is laid 4 inches of Portland-cement concrete, covered with 2 inches of granolithic, composed of clean granite chips and Portland cement, gauged 1 to 1. The cost is 5s. per square yard, and this flooring is found very satisfactory for both light and heavy traffic. The total cost of the buildings averages 3s. per square foot of ground covered. A row of single-storey warehouses has been built opposite the transit-sheds, constructed of iron with party walls of rubble masonry. They cost 0·97d. per cubic foot of contents, or 2s. per square foot of ground covered. In addition, there is a five-floor warehouse at Victoria Dock, with a

Fig. 373.—Shed and Warehouse at Dundee.

* Buchanan on "The Port of Dundee," *Min. Proc. Inst. C.E.*, vol. cxlix.

capacity of 270,000 cubic feet, which cost £16,147 to build." The trade at Dundee is largely in Indian jute, a bale of which measures 4 feet by 1 foot 6 inches by 1 foot 9 inches, and weighs 400 lbs.

Warehouses at Greenock.*

"On the south side of the James Watt Dock, at the east end, a block of warehouses, 676 feet in length, has been erected, one warehouse being 275 feet long and one 223 feet long, 106 feet wide and 47 feet high, and two warehouses 89 feet long, 106 feet wide and 57 feet high to eaves and 97 feet to ridges of roofs. The fronts of the warehouses are constructed of cast-iron columns and girders, with wrought-iron sliding doors, and the back, end, and division walls are of brickwork. The two longer warehouses are arranged with two floors above quay level. The two shorter warehouses have four floors above the quay level; and the insertion of an intermediate floor between the ground and first floors has been provided for. The first floor is made fireproof with 10 by 6 inches rolled beams, spaced $4\frac{1}{8}$ feet apart, carrying brick arches $4\frac{1}{2}$ inches deep at the crown, on top of which the floor is rendered with 1 to 1 Portland cement granolithic composition 1 inch thick. The upper floors and roofs are of timber. A well-hole in the centre of each of these warehouses, 24 feet long by 16 feet wide, enables cranes placed on the top floors to load or unload goods from any floor into or out of railway waggons on the ground floor. The warehouses are intended for general merchandise, but the columns carrying the floors have been cast with openings fitted with valve-flaps, in order that they may be utilised as ducts for distributing grain over any portion of any floor, and also for transferring it from a higher to a lower floor, or for loading railway waggons inside the warehouses. In each floor there are openings, with branch pipes connected to the vertical columns, for receiving the spouts of portable hoppers when it is desired to transfer grain to a lower level, but cover-plates ordinarily close the openings in the floors.

"There are doorways on the ground floor through the brick party walls separating the warehouses, for railway waggons to pass, closed by double iron doors, separated by an air-space of $9\frac{3}{4}$ feet. In front of the warehouses there is a covered way, $27\frac{1}{2}$ feet wide, half outside and half inside the warehouses, enabling loading and unloading to be carried on under cover. This corridor outside the line of warehouses is covered with a fireproof floor similar to that of the warehouses and at the same level, forming a continuous platform, 13 feet wide, in front of the warehouses, on which goods are landed and conveyed into any of the warehouses, thereby enabling imports to be dealt with on the ground floor and exports on the upper floor, and thus admitting of the loading or unloading of the vessels to be largely done by gravitation."

* Kinipple on "Greenock Harbour," *Min. Proc. Inst. C.E.*, vol. cxxx.

Sheds at Glasgow.*

"Except where open quays are necessary, all the quays are lined with excellent modern sheds. These sheds are generally single-storeyed and 60 feet in width, but at the Prince's Dock two-storeyed sheds have been provided, one 1,664 feet long by 70 feet wide, and four of an aggregate length of 5,312 by 75 feet wide. The sheds are placed usually 15 or 20 feet back from the face of the quay. The total floor area provided by the single-storeyed sheds amounts to 111,432 square yards, and by the two-storeyed sheds, to 113,292 square yards, or about 46½ acres in all."

Warehouses at Manchester.†

Two blocks of seven-storey warehouses, situated on the north side of No. 8 Dock, cover an area of about 6,000 square yards. Their concrete foundations rest on hard gravel at depths varying from 12 to 19 feet below quay level. Each warehouse measures about 60 feet by 54 feet, and is divided from its neighbour by a strong party wall. The walls are 3 feet thick at the ground level and 1 foot 10½ inches at the summit. They are carried

Fig. 374.—Shed at Manchester.

6 feet above the roof line. The columns are cast iron throughout, and range from 12 inches diameter and 1½-inch metal, with bases 3 feet square at the basement level, to 7 inches diameter and 1-inch metal at the top floor, carrying the roof 65 feet above ground level. All floors are framed with 14 by 12 inches pitchpine beams, spruce pine joists, and double boarding. The doors and door frames throughout are of iron.

A section of a recently-constructed transit-shed with five floors is shown in fig. 374. The columns and girders are of iron and the floors of concrete.

* Alston on "The River Clyde and Harbour of Glasgow," *Int. Eng. Conf.*, 1901.

† *Engineer*, July 30, 1897.

Fig. 375.—Shed at Manchester.

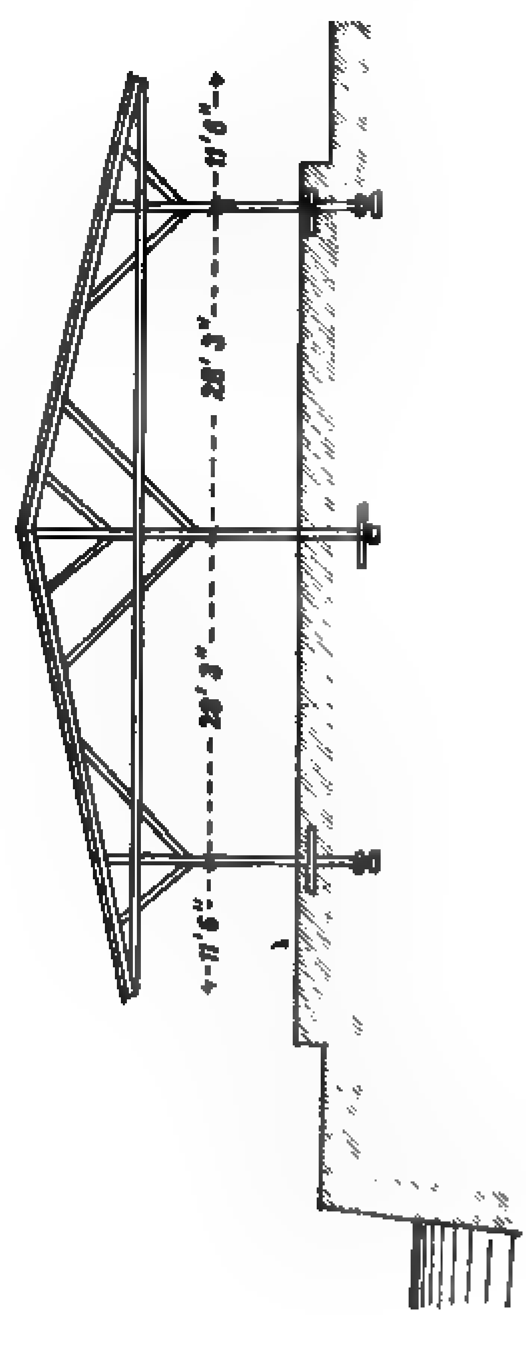


Fig. 376.—Old Shed at Antwerp.

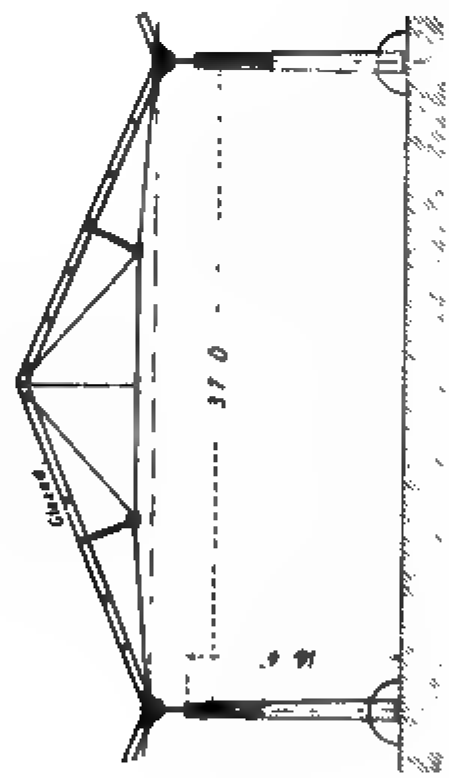


Fig. 377.—Modern Shed at Antwerp.

The balconies on the dockside and roadside are hinged so as to be turned up or let down at will. The topmost floor is unsheltered and is used as an open quay space, upon which goods unaffected by the weather are deposited. The section through a similar shed is given in fig. 375.

Antwerp Sheds.

The older sheds at this port are mainly constructed in timber, having uprights, framing, and roof trusses of red pine with a covering of corrugated iron. One of these sheds is shown in section in fig. 376.

The later sheds along the quays of the Scheldt are entirely constructed in iron. The struts and chairs for roof trusses and the column guards are cast; all the remainder is wrought. The sheds are disposed in groups of several spans, each of a uniform width of 40 feet, with their gable ends facing the river bank. The spaces between the groups range between 40 and 80 feet in width, and are occupied by one or more lines of rails connected with the quay service by means of turn-tables. The depth of the sheds varies from 100 to 160 feet, and they cover an area of nearly 17 acres.

The type of shed is uniform throughout and is illustrated in fig. 377. The roof trusses are situated at 11 feet 4 inches centres, bearing on longitudinal plate girders, 20 inches deep, which span the distance, 34 feet, between consecutive columns. These last are built of two channel irons connected by plates, so as to form a hollow rectangular interior, which is utilised to accommodate the rain-water spouts. The column bases are bolted down to a masonry foundation. The principal rafters are of joist iron; the struts of cast iron, cruciform section; the ties of round iron, and the purlins of angle iron. Along the ridge on the north slope of the roof runs a continuous skylight, 7 feet in width.

Warehouses and Sheds at Rotterdam.

The oldest type of warehouse, constructed in the seventies of last century, has a length of 656 feet and a width of 120 feet. It is divided into five compartments by fireproof partitions, which project beyond the face and above the roof of the building by 6 feet 6 inches. The ground floor and its exterior platform are 3 feet 6 inches higher than the quay level. There are fireproof cellars with an area of 5,330 square yards, and, in addition to this, there are three floors. Along the first of these runs a balcony 23 feet wide; above there is a narrow gangway of 6 feet 6 inches in width.

Contemporaneously with this warehouse were constructed eight sheds, entirely in wood, with an internal width of 59 feet. The floor rests upon piles, spaced 8 feet apart, which, however, are a cause of inconvenience from their continuous settlement and the consequent necessity for raising the floor. The roof covering is bituminous paper (*papier-bitume*) laid on

boarding. It is worthy of mention that, with a single coat of tar per annum, this covering has remained intact for more than twenty-five years. It is, however, very inflammable, and, taken in conjunction with the fact that there is an open void of some 10 feet between the shed floor and the ground, these sheds must be considered constructed in such a manner as to be highly combustible. In fact, one of them was totally burnt in 1889. The reconstruction was carried out entirely in brickwork and iron.

The latest sheds at this port have a width of 131 feet and a length of 357 feet 6 inches. The gable walls and the division wall between the two compartments into which the sheds are divided are of brick, but the remaining sides and the roof are of corrugated iron. The wood floor rests directly on the sand. There are platforms 13 feet in width at the front and 4 feet in width at the back. The roof truss is of the bowstring type, in three spans supported by columns in lattice-work.

Sheds at Havre.*

On the north quay of the Bellot Basin there are three sheds, each 147 feet 6 inches wide, exclusive of overhang, with lengths of 255 feet, 457 feet, and 306 feet respectively. On the south quay of the same dock the sheds (fig. 378) are 180 feet wide and 262 feet, 525 feet, and 590 feet long

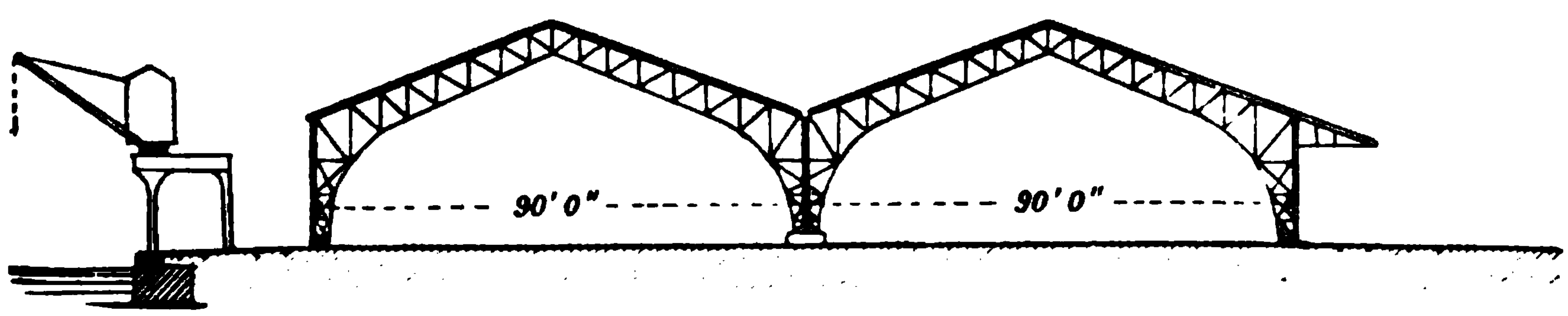


Fig. 378.—Shed at Havre.

respectively. In each case they are separated by open spaces of 130 feet. These spaces are intended not only for the purpose of isolating conflagrations, but also in order to accommodate cumbersome merchandise, and to permit of trucking from the dock quays without the necessity of passing through the sheds. These last have metallic frames, roof coverings of zinc sheets, and external walls of brickwork.

The roofs are in two spans each, of 73 feet 9 inches and 90 feet respectively. The total height of the north sheds is 38 feet and of the south sheds 41 feet. There are continuous doors along the quay front of a uniform height of 15 feet 6 inches in both cases.

Sheds at Marseilles.†

The double-storey shed illustrated in fig. 379 has a roof in one span of 78 feet 9 inches, the ridge of which is 43 feet above ground-floor level. The

* Despres on "The Plant of Maritime Commercial Ports of France," *Proc. Am. Soc. C.E.*, vol. xxx.

† *Ibid.*

upper floor, supported on cast-iron columns 20 feet apart, is placed at a height of 16 feet, and is extended so as to form an exterior gallery, 11 feet 9 inches in width. Examples of single-storey sheds are given in fig. 380.

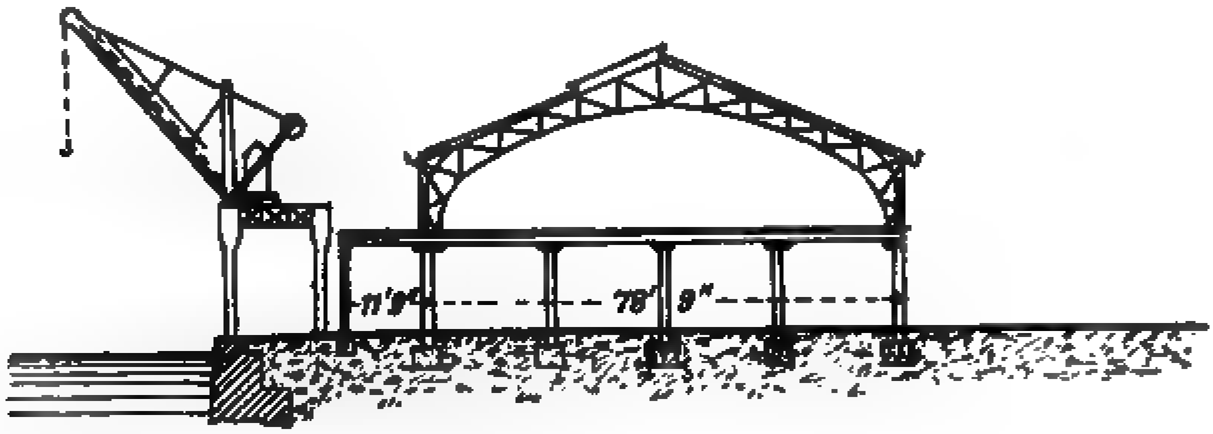


Fig. 379.—Shed at Marseilles.

Fig. 380.—Sheds at Marseilles.

The following are stated to be the dispositions found to be most suitable for sheds at this port:—

The shed to be enclosed on three sides; on the fourth, or dock, side to have doorways alternating with solid panels. Sliding doors in two leaves, with angle-iron frames, iron sheeting, and wood border. Roofs, in spans not exceeding 100 feet in width, carried on cast-iron columns, serving as downspouts. Trusses, 16 feet apart, with framed iron principals and wood purlins. Roof covering of tiles, with a double lantern, 12 feet wide, on each slope, astride ridge.

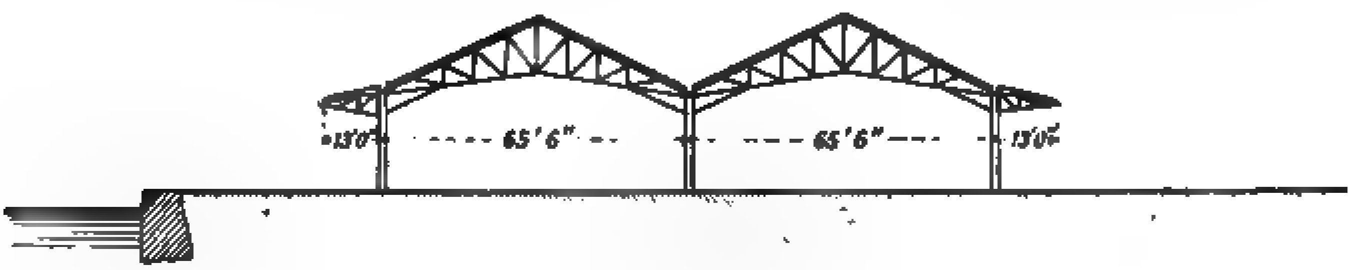


Fig. 381.—Shed at Calais.

Other French Ports.

The sheds at *Calais* (fig. 381) are in two equal spans of 65 feet 6 inches, with overhangs of 13 feet on each side.

The sheds at *Dunkirk* (fig. 382) are in one span of 98 feet 6 inches, with a short overhang on the roadside.

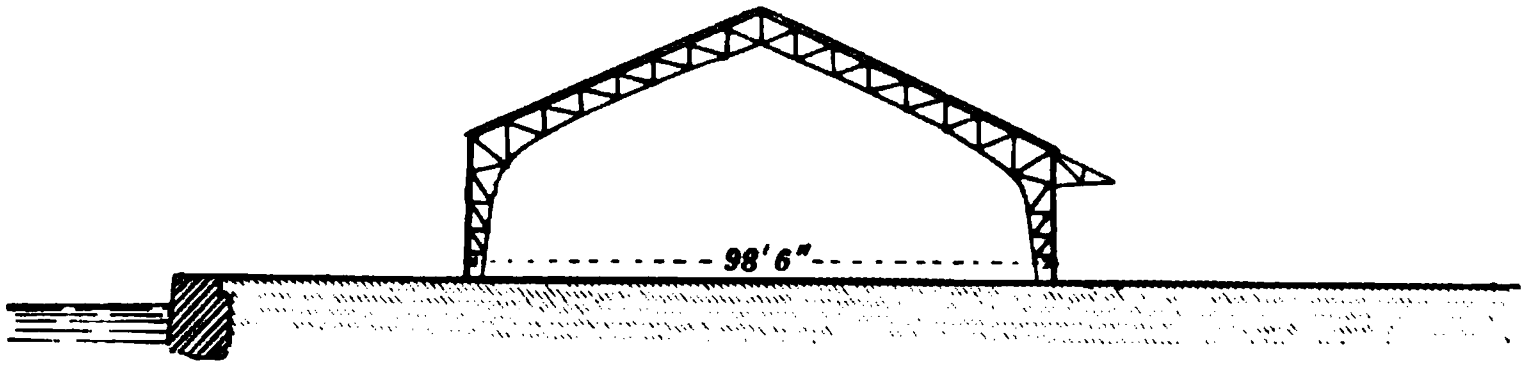


Fig. 382.—Shed at Dunkirk.

At *Dieppe* (fig. 383) and at *Rouen* (fig. 384) the spans are 78 feet 9 inches and 82 feet 6 inches, and the overhangs 11 feet and 14 feet 9 inches respectively. At *Bordeaux* the span is 65 feet.

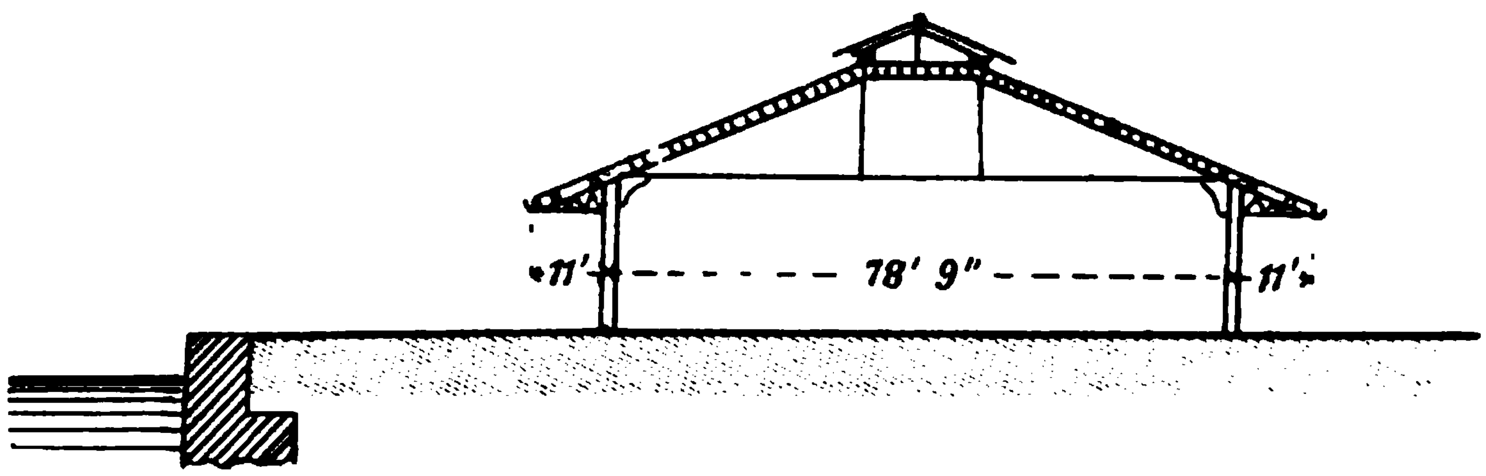


Fig. 383.—Shed at Dieppe.

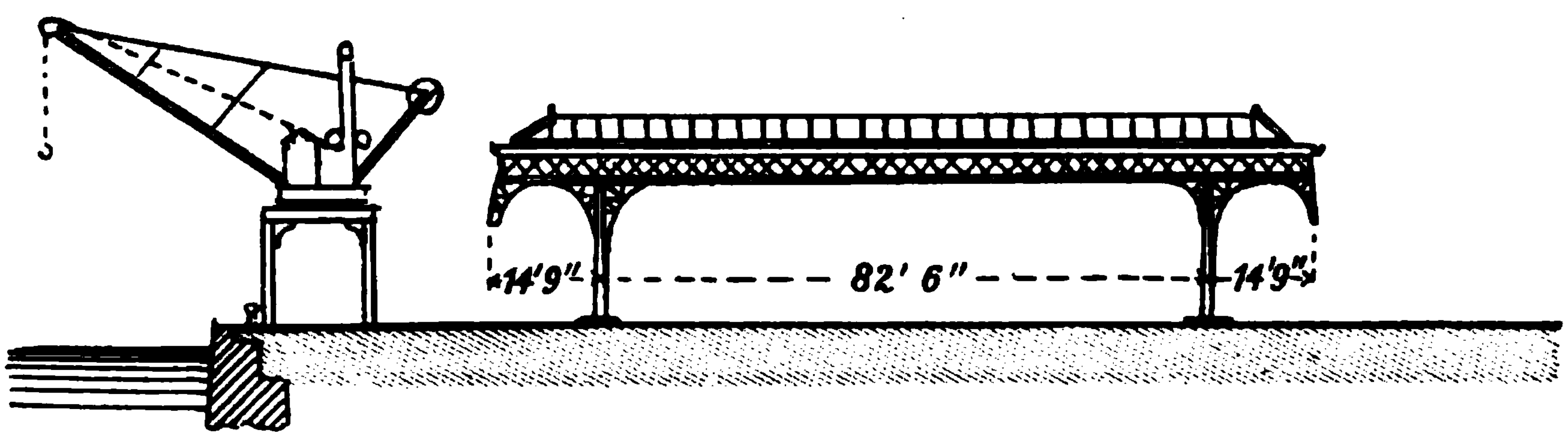


Fig. 384.—Shed at Rouen.

At French ports the practice is generally to locate the sheds, so that a distance of 30 to 40 feet separates them from the edge of the quay.

Sheds and Warehouses at Bremen.*

"The fronts of the quay sheds, which are for the most part 131 feet wide, are entirely closed by galvanised corrugated iron sliding doors, so that several hydraulic cranes can be worked together, and the vessel can be unloaded from several holds at the same time. A shed can, therefore, be entirely closed or opened on the water side, and on the land side, access is given by doors, between each two of which a crane is placed. These sheds are surrounded by loading stages, and, in order that the cart traffic may be kept separate from the railway traffic, they are arranged so that vehicles

* Franzius and De Thierry on "River, Harbour, and Canal Works in Germany," *Min. Proc. Inst. C.E.*, vol. cxxxv.

Fig. 385.—Quay Sheds and Warehouse at Bremen.

Fig. 386.—Quay Sheds and Warehouse at Bremen.

Figs. 387 and 388.—Sheds at Hamburg.

may drive in under them from the street, and nine may at one time be conveniently loaded from the floor level, which is the same as that of the loading stages. Besides this, the warehouse fronts serve the vehicular traffic. On the rebuilding of one of these sheds, which had been completely destroyed by fire in a short time, it was divided up by two fireproof walls. The total length of sheds already built (in 1898) amounts to 5,052 feet, and they have a total area of 724,800 square yards. For unloading and storing cargoes of cotton, there is a shed on the north side of the dock, the floor of which is at street level on the water side, and rises gradually to the level of the loading platforms on the land side. This shed, built of wood and corrugated iron, and roofed with roofing paper, differs from the others, which are built of iron. Behind this shed is a storage warehouse, which is built in a similar manner.

"Two grain warehouses were erected in 1896-97. One warehouse, on the quay, 558 feet long and 135 feet wide, has only one storey for the first third of its width towards the water side, while the remaining two-thirds are two storeys high. At the back of this warehouse, and separated by a street 66 feet wide, down which lines of railway pass, is a two-storey warehouse 886 feet long and 98 feet wide. The upper floors of both warehouses are intended chiefly for grain in bulk. The warehouse on the quay covers an area of 7,940 square yards, and the storage warehouse 6,110 square yards. They can store 18,000 tons in a manner usual for a lengthy period, and 12,000 tons for the time usually adopted for grain in towns. The total cost amounted to about £56,100."

A section across Bremen Quays is given in figs. 385 and 386.

Sheds at Hamburg.

The sheds on the quays (figs. 387 and 388), where the sorting of the unloaded goods is generally done, are one storey high throughout; they are closed on the land side and open to the water. On the land side there are four or five lines of railway, on the two first of which trucks stand to be loaded. The goods unloaded from the sea-going ships, which are to be forwarded by rail, are dealt with on the land side of the sheds, while those to be sent to warehouses in the town by barge are dealt with by cranes on the water side. The water side is paved throughout, forming a roadway for vehicles. With the exception of those on the Sandthor Quay, all sheds are built of wood and roofed with roofing paper. The Sandthor Quay sheds have stone walls on the land side and are roofed with iron. The breadth of the sheds varies between 48 feet, on the Sandthor Quay, and 110 feet, on the Asia Quay.

Kidderpur Dock Sheds, Calcutta.*

"Cargo sheds have been constructed on both sides of the dock, each shed being 300 feet long by 120 feet wide. They are constructed in two

* Bruce on "The Kidderpur Dock Sheds, Calcutta," *Min. Proc. Inst. C.E.*, vol. cxxi.

bays of 60 feet each, carried on cast-iron columns of **H** section, and are roofed with corrugated iron, and are enclosed by 15-inch brick walls built between the **H** columns, and fitted with sliding doors, one in each 15-foot bay. To avoid down pipes passing through the sheds to drainage channels under the floors, the centre gutters are made large enough to carry the rain-water to the ends of the buildings, the necessary fall being obtained by raising the bases of the middle columns. All the gutters are of $\frac{1}{8}$-inch galvanised steel plates. The floors are laid with a slope of 1 in 60, and, on the quay side, are raised 1 foot above the coping level. The height above ground level at the back of the shed is 3 feet 6 inches, and along this inner face a platform, 8 feet wide, has been constructed for the convenience of the railway traffic. The total shed area provided is 432,000 square feet. The sheds are lighted by electricity. Each of the sheds has forty 16-candle-power incandescent lamps, hung from the tie-beams of the principals. In addition to these, and to arc lamps upon the quays, a terminal box is provided in each shed, to which a portable lamp may be connected, in case of more light being required in any part of the shed, or outside, when loading or unloading has to be carried on at night."

Sheds and Warehouses at Buenos Ayres.*

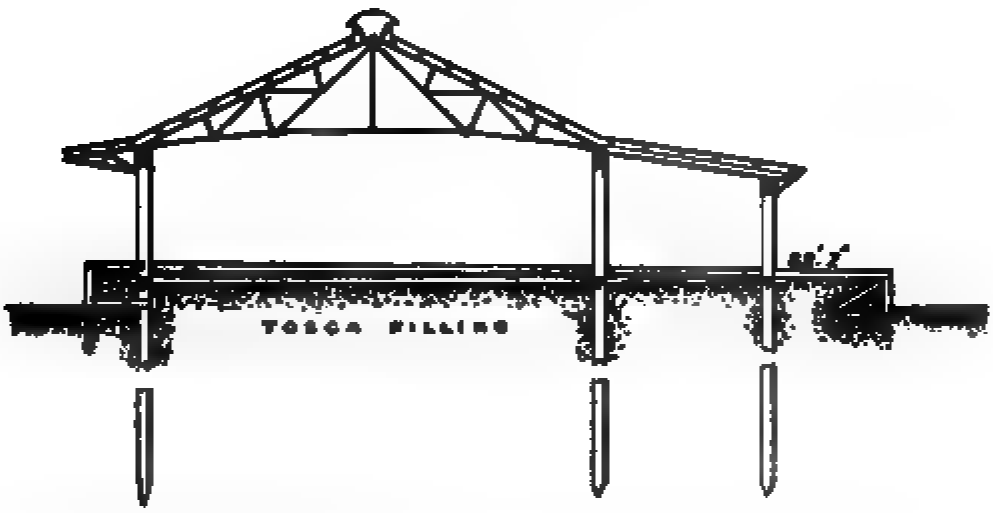
The total capacity of the sheds and warehouses amounts to 687,378 cubic yards, and the total floor area to 230,595 square yards. The sheds are of iron, with corrugated iron roofs. They are mainly built on piles in made ground. Each shed has a platform, 31 feet wide, on the dock side, covered by a verandah.

Four of the thirteen warehouses have wooden roof-trusses, with tiles laid on planking. The remainder have iron roof-trusses, with a zinc covering, as well as iron partition doors and iron window frames. These latter warehouses have an extra floor, making five in all, and have longitudinal platforms running the whole length of the front of the warehouses, so that goods can be deposited on any part of the platforms in order to be removed into the warehouses afterwards. The warehouses are built of rubble masonry up to the level of the quays, and from that level to the top, of brickwork. All the floors are of timber, with the exception of the ground floors over the bonds, which are of concrete in the proportion of 1 cement, 4 sand, and 6 stone, with a rendering of 1 inch of 1 cement and 1 sand.

Figs. 389 to 391 illustrate the practice at this port.

Various other instances of shed and warehouse construction, sufficiently intelligible without description, will be found in figs. 392, 393, and 394, which illustrate sheds at **Zeebrugge** and **Emden**, and a warehouse at **Amsterdam**.

* Dobson on "Buenos Ayres Harbour Works," *Min. Proc. Inst. C. E.*, vol. cxxxviii.



QUAY SHEDS

QUAY SHED

WAREHOUSE.

Figs. 389, 390, and 391.—Sheds and Warehouse at Buenos Ayres.

Fig. 392.—Shed at Zeebrugge.

Fig. 393.—Shed at Emden.

Fig. 394.—Warehouse at Amsterdam.

CHAPTER X.

DOCK BRIDGES.

CLASSIFICATION—FLOATING BRIDGES—TRAVERSING BRIDGES—DRAWBRIDGES—BASCULES — LIFTING BRIDGES — SWING BRIDGES — SINGLE-LEAF *versus* DOUBLE-LEAF BRIDGES—STRESSES IN MOVABLE BRIDGES—CASE OF THE DOUBLE CANTILEVER—CASE OF THE CANTILEVER AND BEAM—CASE OF THE ARCH—CASE OF THE CONTINUOUS BEAM—THE THEOREM OF THREE MOMENTS—EFFECT OF COUNTERPOISE—LOADS IMPOSED ON MOVABLE BRIDGES—WEIGHT OF STRUCTURE—WEIGHTS OF TYPICAL LOCOMOTIVES—EQUIVALENT LIVE LOADS—WEIGHT OF VEHICLES AND MEN — PRACTICAL EXAMPLE OF THE CALCULATIONS FOR A SWING BRIDGE — DISTINCTIVE FEATURES OF MOVABLE BRIDGES—THE PIVOT—BALANCED ROLLERS AND WHEELS—THE COUNTERPOISE—SETTING APPARATUS—INTERLOCKING APPARATUS—NOTES ON DESIGN—ILLUSTRATIONS OF MOVABLE BRIDGES AT GREENOCK, ANTWERP, ROTTERDAM, CHICAGO, MARSEILLES, LIVERPOOL, LEITH, AND KIDDERPUR.

NARROW waterways and locks, linking together the various parts of a dock system, are generally spanned at convenient points by bridges, in order that vehicular and foot traffic may be transmitted across them and access provided, as uninterruptedly as possible, to all quarters. On account, moreover, of the necessity of maintaining the navigation of these passages, it is essential that bridges crossing them should be of a movable nature and characterised by great rapidity of action, so as to avoid lengthy stoppages and interference with the use of either road or waterway.

Such bridges are, of course, used in a variety of situations and in branches of engineering not necessarily connected with docks. Their importance, however, to the dock engineer is indisputable.

Classification.—For the purpose of this treatise, movable bridges may be divided into five classes:—

Floating bridges.
Traversing bridges.
Drawbridges.
Lifting bridges.
Swing bridges.

Floating Bridges, as the name implies, are water-borne, either continuously and wholly, or partially and during such times as they are being moved. The former variety, which are generally formed of pontoons, either singly or in combination, are rarely used otherwise than for purposes of a purely temporary nature, such as the crossing of rivers and streams during military operations. A striking instance of their application to

more permanent ends is afforded by the Liverpool and Birkenhead landing stages on the River Mersey, which, themselves constructed on the same principle, are connected with the shore by floating bridges, consisting of a series of pontoons, flexibly linked together so that they are able to adapt themselves to the fluctuations of tidal level. The length of the Liverpool bridge is 550 feet and its width 35 feet. The Birkenhead bridge is 678 feet in length by 30 feet in width. Neither of these bridges is, however, a movable bridge in the sense intended in this section.

There is a pontoon bridge, which is movable in the true sense of the word, over the Kaiser William Canal at Holtenau. It consists of two main or turning pontoons, meeting at the centre of the canal, united to two bearing pontoons at their shore ends. The bridge, which carries a 15-foot roadway and two 2 feet 6-inch footpaths, is opened by turning the pontoons round their shoreward ends, and this is accomplished by having a chain, one end of which is attached to a mushroom anchor in the bed of the canal, and the other to a bollard on the bank, wound round the barrel of a winch, which is on a small pontoon alongside of, and fixed to the main pontoon.

The second kind of floating bridge is represented by caissons, which, however, only act incidentally as bridges, their primary function being that of closing a waterway. It has already been noted that one of the advantages appertaining to a caisson, in comparison with a pair of gates, is this capacity to discharge dual duties, whereby the additional expenditure for a bridge is avoided. Caissons as a class have already been dealt with in Chap. viii., so that there is no need to pursue this branch of the subject further.

Traversing Bridges are supported by the quay at or about the coping level and are projected forward or withdrawn in a straight line—in other words, their motion is rectilinear and approximately horizontal, or with just sufficient inclination to enable them to clear the edge of the roadway abutting on their recesses; for, except in the case of footbridges, which may be provided with approach steps at each end of the bridge, forming part of the moving structure, the wheel track of a traversing bridge must lie somewhat below the quay level in order that its floor may form a continuous horizontal plane with the roadways. Consequently, for the purpose of removal, the tail or inner end of the bridge must be raised to the height of the roadway before it can be drawn backwards.

Several arrangements have been devised for the working of traversing bridges, of which the following are a few typical instances:—

(*a*) The nose or forward end of the bridge rests upon rollers driven in between the bridge girders and the wall-bearing plate. In order to open the passage these rollers are withdrawn, and, at the same time, the tail end is lifted. The bridge tilts about intermediate wheels, fixed at the quay edge, and upon these and the tail-end wheels the structure is supported during withdrawal.

(*b*) The same effect of tilting the bridge is obtained by making the tail

end lighter than the overhanging portion. The nose end is then provided with movable supports, and when these are lowered, the bridge naturally inclines into a position suitable for removal.

(*c*) The intermediate support is formed by a pair of wheels surmounting hydraulic rams which lift the bridge bodily. The nose end of the bridge is the lighter end, and is checked in its tendency to rise by a bracket which engages in the abutment. This allows the tail end to clear the roadway prior to being drawn over fixed wheels at its edge.

(*d*) The main girders of the bridge have prolongations in the form of bent levers, inclined upwards and counterweighted, so that, with a slight additional pressure, the inclined tail is brought down to the level of the roadway, and the bridge, with its nose end now tilted, moves backwards over wheel tracks provided for it.

Traversing bridges are much inferior to swing bridges, in that the working friction on the axles is considerably greater than that on a pivot, but they afford decided advantages where it is desirable not to curtail the length of the quayage, since they only occupy a frontage equal to their width. They share this feature in common with the class of bridges next to be considered.

Drawbridges are the most ancient of all movable bridges, dating back to mediæval times, when a militant nobility were in the habit of girdling their residences with moats or ditches, spanned by bridges which could be raised for defence or lowered for sortie, as occasion might require. Such a bridge consisted of a single flap. It was raised by chains attached to the nose end; these passed over pulleys at the summit of uprights fixed near the hinged end.

The later development of this type of bridge is known as a **Bascule Bridge**. Like its prototype, it revolves about a horizontal axis, but it is also provided with a counterpoise in the form of a weighted prolongation of the bridge, whereby the power required for working the bridge is reduced to a minimum. An alternative method of counterbalancing is by means of overhead beams, set a little back from the axis of rotation. The first method needs a deep pit to receive the tail end of the bridge when in the vertical position, and this is not always easy to provide without some portion of the counterpoise becoming submerged. Hence the second method, which is much in vogue in Holland, where the quays are very little above water level.

A third method of counterbalancing the structure is by means of weights attached to chains connected with the bridge and passing over pulleys carried by independent posts. This method has the objection that, the moment of the bridge about its axis being variable at different stages of the lift, while the moment of the counterpoise remains constant, the bridge cannot be maintained in even approximate equilibrium throughout.

A compound arrangement of self-contained and extraneous balancing is afforded by the design in fig. 395, due to Mr. W. R. Browne. The axis of rotation is fixed some little distance away from the centre of gravity of the

bridge, being both horizontally behind and vertically below it. At the instant of commencing to open the bridge, the moment of the counterbalance is slightly in excess of the moment of the bridge, thus assisting it to rise. The excess continues until the centre of gravity of the bridge comes vertically over the axis, at which stage the line of chain also intersects it, producing equilibrium. As the bridge continues its rotation the contrary effect is set up, the moment of the bridge tending to increase its travel, while the moment of the counterpoise acts as a check. In closing the bridge the action is the same, but in reversed order.

Sectional Elevation at outside plummer-block.

Section along centre line of Bridge.

Fig. 395.—Bascule Bridge.

Bascule bridges are usually in two leaves, meeting at the centre of span. The under side of each leaf is then perfectly curved in form, or is provided with raking struts, fitting into pockets or recesses in the side walls when the bridge is lowered. This type of bridge forms an arch, and, accordingly, it derives very considerable support from the mutual abutments at its centre and the skewbacks at the sides. These parts can be adjusted to a nicety which is not realisable in the case of other types.

The main objections to the employment of bascules are their liability to come in contact with the yards and spars of passing vessels, and also the very large surface which they expose to wind pressure. The leverage exerted by the wind materially

increases the labour of raising the bridge, and from the nature of its office no perforations are allowable in the bridge platform. The former drawback can be remedied to some extent by setting back the axis from the face line of the quay, but this step considerably augments the length and cost of the bridge.

It has been proposed* as an antidote to both evils that the bridge, instead of being raised, should be lowered into its vertical position and at the same time recessed within the side walls. The author is unaware of any instance in which the suggestion has been carried out. Except in the case of very high quays the project would evidently entail the submersion of a part of each leaf; but, though this might be detrimental to the durability and appearance of the structure, from an operative point of view it would confer a benefit rather than otherwise. There are one or two obvious difficulties to be overcome, but the author of the scheme (Mr. C. J. Findlay) does not consider them insuperable.

All bascules do not rotate about a fixed axis. There is a variety, known as the *rolling bascule*, in which the tail-end of the bridge takes the form of a circular segment, upon which the bridge rolls in a manner similar to the action of a rocking chair (fig. 435).

Lifting Bridges are horizontal platforms raised vertically in such a way as to maintain a level surface throughout the process. Instances of their use are rare, and are apparently confined to rivers and canals. Indeed, their eligibility for dock work, except, conceivably, in connection with canal basins, is dubious, owing to the great height to which they would have to be raised in order to clear the masts of vessels passing beneath them. Furthermore, it would be a difficult matter to secure equable movement of the platform, lifted, as it would be, from two opposite sides of the waterway, unless the action were controlled from one centre—an arrangement which is scarcely feasible in the majority of cases. The advantages attached to the system are limited to a minimum appropriation of quay space. There is a lifting bridge over a channel 100 feet wide at Chicago.

Swing Bridges.—These constitute by far the most numerous and the most important class for dock work. It includes all movable bridges in which the axis of rotation is vertical. The merits of the principle are a comparatively slight expenditure of motive power, ease of movement, and less wear of the bearing surfaces, the absence of deep pits for counterbalancing purposes, and of appreciable change in level. On the other hand, two important drawbacks must not be overlooked :—

1. Swing bridges are necessarily longer than bascules or traversing bridges. Bascules may rotate about an axis as close to the edge of the coping as is considered desirable. The pivot of a swing bridge must, however, be set back a distance not less than half the width of the bridge, in order that the latter may be entirely housed within the quay line when the passage is open. Since the counterbalance must lie behind the pivot, it necessarily follows that,

* Findlay on "The Design of Movable Bridges," *Min. Proc. L.E.S.*, vol. ii.

in the second case, both the effective span and the length of tail are increased. This consideration is not without importance on grounds of economy alone. In traversing bridges the effective length is measured between the bearings, and these may be practically at the edge of the coping.

2. The side recesses of swing bridges occupy a large extent of valuable quay frontage—much more than other kinds of bridge—and in the case of wide passages this leads to the necessity for side walls of considerable length, with a corresponding increase in cost of construction, apart from any question of intrenching upon the area of dock accommodation.

On the whole, the balance of technical opinion, as evidenced by practice, inclines toward the employment of swing bridges in preference to other types, in so far as heavy traffic, at any rate, is concerned. There is one case in which a swing bridge offers signal advantages. When two waterways of about equal width lie side by side with an island between, as in fig. 456, a swing bridge may be arranged symmetrically upon a central pivot, so that each wing acts as a counterbalance to the other. In this way the length of bridge necessary for closing the opening is reduced to a minimum, and the rotation of the wings neutralises the effect of the wind pressure upon the surface of the bridge. This last statement, though theoretically convincing, is only partially true in practice, for, as has already been pointed out, the wind does not exert a uniform pressure over large areas. On the contrary, it is given to surging and eddying; consequently it is quite possible that, however symmetrically disposed the wings of a bridge may be, the pressure on one will exceed that on the other. Such was proved to be the case by an incident at Goole, where a bridge over the River Ouse spanning two openings, each 100 feet wide, broke loose in a severe gale and swung violently backwards and forwards, describing about one-third of a circle each time.

The converse of a single bridge spanning two openings is that of a bridge in two leaves spanning a single opening, and we now enter into the merits of single-leaf and double-leaved bridges respectively.

Single-leaf *versus* Double-leaf Bridges.—The relative advantages and disadvantages of single and double leaves, in the cases of traversing, bascule, and swing bridges, may be summed up as follows:—

1. The depth of a single-leaf bridge is necessarily greater than that of a double-leaf bridge for the same span. If s be the span, w the weight per foot run, t the maximum permissive tension in the top flange—all fixed values—we have, by taking moments about the bottom of the bridge at the side of span—

$$(a) \text{ in the case of a single leaf, } d_1 = \frac{w s^2}{2 t}$$

$$(b) \text{ in the case of double leaves, } d_2 = \frac{w s^2}{8 t},$$

so that the depth (d) of the bridge, treated while in motion entirely as a cantilever, needs to be four times as great in the first case as in the second.

If we consider the support afforded by the further abutment in the single-leaf bridge when at rest, the ratio is, of course, considerably reduced.

2. On the other hand, the length of a single-leaf swing bridge is less than the combined lengths of two leaves for the same opening. The reason for this has already been given—viz., that the pivot has to be placed sufficiently far back from the face of the coping to accommodate the whole width of the bridge upon the quay. When there are two pivots, the excess of length thus involved is doubled.

3. A single-leaf bridge only requires a single set of actuating machinery. For a given opening, the set will have to be at least twice as powerful as the two sets combined, but the cost of construction, of repairs, and of general maintenance will certainly not be doubled for a single set.

4. The control of the machinery of a single-leaf bridge is in the hands of one man. Two sets of machinery necessitate two attendants at least, whose co-operation can only be secured by imperfect signals or by shouted instructions, which, in windy weather, are liable to be unheard or misunderstood.

5. Additional apparatus for interlocking is required in the case of double-leaved bridges.

6. The adjustment of the levels of two leaves at their junction is a matter of some delicacy. Any irregularity (however slight) in the joints of a locomotive track leads to percussive action and the gradual destruction of the rail. The absolutely necessary clearance between the two sets of rails is sufficient to cause this, and repairs or renewal involve inconvenience and delay.

7. A double-leaf swing bridge necessitates less length of passage than a single-leaf bridge, the whole of whose length has to be accommodated on one side.

Stresses in Movable Bridges.—It would manifestly be impracticable, within the limits of a single chapter, to attempt to treat with the least degree of precision and finality the very numerous and important considerations peculiarly involved in the design of movable bridges. Still less would it be possible to investigate, with that thoroughness which the question demands, the nature and amount of the stresses set up in the framework of such bridges, generally, under the varying conditions of load and support to which they are subjected. These latter problems form the basis of distinct treatises, to which the reader is referred for information more complete, more detailed, and more comprehensive than could be included here.

At the same time, in view of the identification of movable bridges with dockwork and the unique features which they possess in that connection, it would evidently be equally injudicious and inappropriate to abstain altogether from presenting some account of the principles, upon the basis of which such structures are adapted to the particular kind of work which they are called upon to perform.

Accordingly, we will endeavour to compromise the matter by investiga-

ting, in the first instance in general terms, and then, as far as practicable, in some brief detail, the fundamental problems which present themselves to the engineer in designing movable bridges, in so far as they are connected with the equipment of docks.

Excluding floating and lifting bridges as too remotely related to the subject for general application, we may divide our consideration of the stresses in the remaining kinds of bridge into four cases, representing the different conditions in which any of them may be found.

(1) A double cantilever resting upon a central support.

(2) A single cantilever supported at two points, or a cantilever and beam combined.

(3) An arch.

(4) A continuous girder resting upon three supports.

The first case represents a single swinging bridge with two equal wings. The second embraces generally all cases of bridges in one or two leaves projecting over an opening, with certain exceptions, as under. The third applies to those double-leaved bridges which afford one another mutual support at their meeting faces; and the fourth is the normal condition of a single-leaf bridge in a state of rest.

Case I. A double cantilever resting upon a central support (fig. 396).

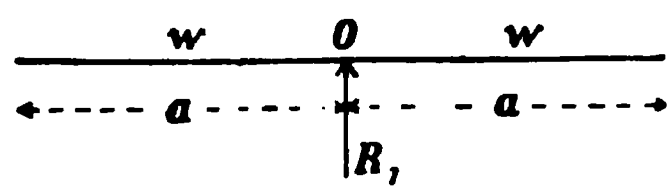


Fig. 396.

This is an extremely simple case and need not be the cause of more than a moment's detention in passing. If the imposed load be w per foot run, and, assuming that the bridge is symmetrical, the central reaction is obviously—

$$R_1 = 2\,w\,a, \quad . \quad . \quad . \quad . \quad . \quad (81)$$

and the shearing stress increases from zero at each extremity to one-half of the above amount on each side of the support.

The bending moment at O is—

$$M_0 = \frac{w\,a^2}{2}, \quad . \quad . \quad . \quad . \quad . \quad . \quad (82)$$

and at each end it is zero. The curve of moments for each half of the bridge is parabolic, with its origin at the extremity and its axis vertical. Where the arms are of unequal length, the stresses are clearly those due to the longer arm, a counterpoise being added to the shorter arm to produce an equal effect.

Case II. A single cantilever in combination with a beam (fig. 397).

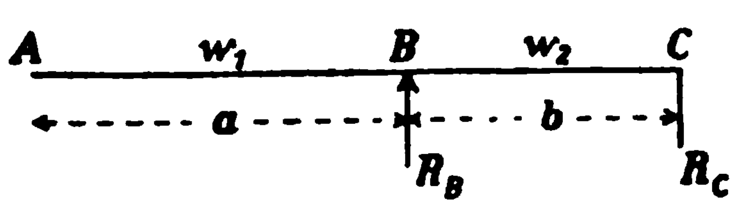


Fig. 397.

Let A C be a girder of total length $(a + b)$ supported at two points, B and C, of which only one is at an extremity.

The reactions, R_B and R_C, at B and C may be determined by taking moments about the points C and B, respectively, thus—

$$R_B b = w_1 a\left(\frac{a}{2} + b\right) + w_2 \frac{b^2}{2} \quad . \quad . \quad . \quad . \quad (83)$$

$$R_C b = w_1 \frac{a^2}{2} - w_2 \frac{b^2}{2}, \quad . \quad . \quad . \quad . \quad . \quad . \quad (84)$$

R_C being measured downwards. The amount of counterpoise required to prevent the cantilever end overbalancing is, accordingly, the positive term in the value of R_C in (84)—viz., $\frac{w_1 a^2}{2b}$.

At any point distant x to the left of B, the shearing stress is—

$$S_1 = w_1 (a - x), \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (85)$$

and the bending moment—

$$M_1 = w_1 \frac{(a - x^2)}{2}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (86)$$

These become $w_1 a$ and $\frac{w_1 a^2}{2}$, respectively, at B.

At any point distant x to the right of B, the shearing stress is—

$$S_2 = w_2 (b - x) + R_C$$

$$= w_2 (b - x) + w_1 \frac{a^2}{2b} - w_2 \frac{b}{2}, \quad . \quad . \quad . \quad . \quad (87)$$

and the bending moment—

$$M_2 = \frac{w_2 (b - x)^2}{2} + R_C (b - x)$$

$$= \frac{b - x}{2}\left[w_2 (b - x) + w_1 \frac{a^2}{b} - w_2 b\right]$$

$$= \frac{b - x}{2}\left[w_1 \frac{a^2}{b} - w_2 x\right]. \quad . \quad . \quad . \quad . \quad (88)$$

At B, these become $w_2 \frac{b}{2} + w_1 \frac{a^2}{2b}$ and $\frac{w_1 a^2}{2}$, respectively.

The same equations necessarily hold good whether a closed cantilever bridge be supported at one point by the pivot, or by bearing blocks located nearer the edge of the quay, the only difference being in the respective lengths of the two portions of the bridge. The general practice is to raise the tail end of the bridge with wedges, screws, rams, or other contrivances, so as to throw the forward pressure on to bearing blocks and relieve the pivot and rollers of unnecessary stress. In this way the length of the overhanging or cantilever portion of the bridge is reduced, and it is even possible that the reduction in length of the closed bridge may more than compensate for the increased load which it incurs in that position.

When the bridge is swinging the pressure on the pivot is that due to the ordinary weight of the structure plus the counterpoise, which, computed to balance the bridge under the condition of maximum load, generally throws some excess of pressure upon the rollers.

To find the amount of the respective pressures on the rollers and the pivot, let P (fig. 398) be the position of the pivot and C that of the rollers.

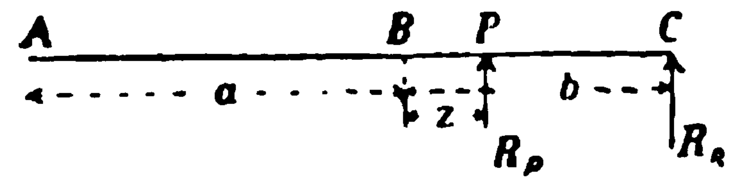


Fig. 398.

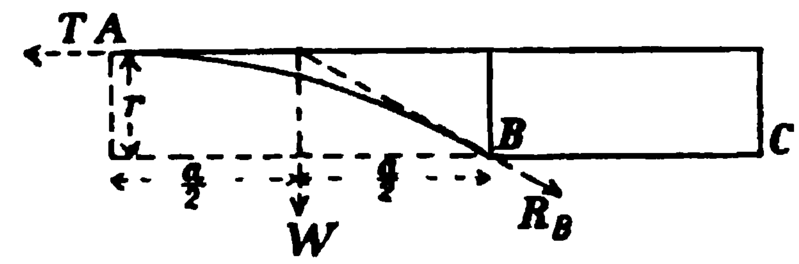


Fig. 399.

Then taking moments about P—

$$\frac{w\,(a+z)^2}{2} = \frac{w\,(b-z)^2}{2} + \frac{w_1\,a^2\,(b-z)}{2\,b} - R_R\,(b-z),$$

whence,

$$R_R = \frac{w}{2}\left\{\frac{(b-z)^2-(a+z)^2}{(b-z)}\right\} + \frac{w_1\,a^2}{2},$$

and

$$R_P = W + C - R_R, \quad . \quad . \quad . \quad . \quad . \quad . \quad (89)$$

where $W = w\,(a+b)$ and $C = \dfrac{w_1\,a^2}{2\,b}$.

Case III.—An Arch (fig. 399).—In the preceding investigation each wing of the closed bridge has been treated separately. If it be desired to take advantage of the support afforded by the mutual abutment of the two meeting faces of the bridge, it is evident that the tail end must be lifted in order to develop the full thrust due to the dead weight. Assuming (as would essentially be the case) that the underside of the cantilevers constitute a real or virtual arc of rise r and span $2\,a$, we have by the conditions of equilibrium for three forces and by similar triangles,

$$\frac{T}{W} = \frac{a}{2\,r},$$

or, approximately,

$$\frac{T}{w\,a} = \frac{a}{2\,r},$$

whence,

$$T = \frac{w\,a^2}{2\,r}, \quad . \quad . \quad . \quad . \quad . \quad (90)$$

which gives us an expression for the amount of mutual thrust. The upward force, F_C, at C, required to develop this thrust is found by taking moments about B—

$$T\,r = F_C\,b,$$

so that,

$$F_C = \frac{w\,a^2}{2\,b}, \quad . \quad . \quad . \quad . \quad (91)$$

b being the distance, BC. This force, the value of which is identical with that of the counterpoise, is additional to the reaction at C, due to the load on BC.

The pressure on the abutment B is

$$R_B = w\,a\sqrt{1 + \frac{a^2}{4\,r^2}}; \quad . \quad . \quad . \quad (92)$$

from which it is apparent that it may be considerable and that carefully adjusted and solid bearings are essential. It is a matter of some difficulty to secure these in the case of swing bridges, and accordingly it is not usual for the central reaction to be much, if at all, relied upon. In bascule bridges, on the other hand, it is comparatively easy to provide accurate bearing surfaces.

Case IV.—A continuous beam supported at three points (fig. 400).—Let

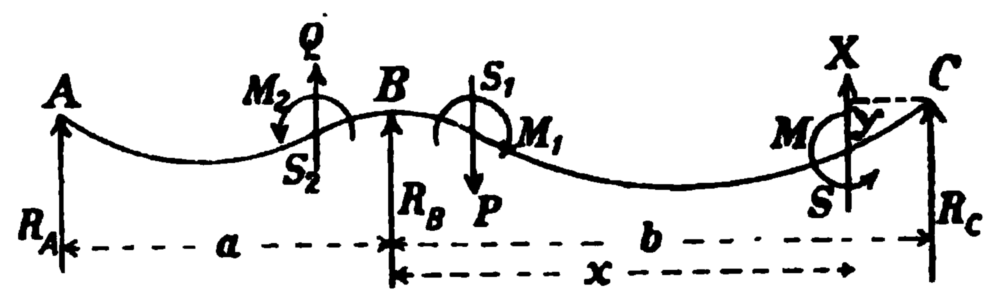


Fig. 400.

A B C be a girder continuous over three points of support—A, B, and C all on the same level. Take the intermediate support, B, as the origin of co-ordinates, and let y represent the deflection of the beam at the point X due to a uniform load, w, per unit length. Let S be the shearing stress and M the bending moment at the same point.

By a well-known formula establishing the connection between the bending moment (M) the modulus of elasticity (E) the moment of inertia (I) and the radius of curvature (R), we have at any point X—

$$M = \frac{E\,I}{R}.$$

Now, when the curvature is very small, as is assumed to be the case in the foregoing relationship, we may find a very close approximation for the value of $\frac{1}{R}$ from the principles of the Calculus, viz.:—

$$\frac{1}{R} = -\frac{d^2 y}{d\,x^2},$$

where x and y are the co-ordinates of the deflection curve. Hence we may write—

$$M = -E\,I\frac{d^2 y}{d\,x^2}.$$

Again, let us consider the conditions of equilibrium at the point X. If P be a point indefinitely near to B, where the shearing stress is S_1 and the bending moment M_1, it is clear that for equilibrium of the portion P X, we have—

$$M = -M_1 + S_1\,x + \frac{w\,x^2}{2}. \quad . \quad . \quad . \quad . \quad (93)$$

Equating the two values of M, we obtain

$$\mathrm{EI}\frac{d^2 y}{d x^2} = \mathrm{M}_1 - \mathrm{S}_1 x - \frac{w x^2}{2};$$

whence, integrating,

$$\mathrm{EI}\frac{d y}{d x} = \mathrm{C}_1 + \mathrm{M}_1 x - \frac{\mathrm{S}_1 x^2}{2} - \frac{w x^3}{6}. \quad . \quad . \quad . \quad (94)$$

To find what value to attach to the constant (C_1) in this expression, we have the following consideration:—Let β be the slope of the beam at the origin, B—or, in other words, the inclination of the tangent of the curve to the horizontal. Then $\tan \beta = \frac{d y}{d x}$ and, in the limit, $\tan \beta = \beta$. When this is the case x is so small as to become negligible, and so we can write

$$\mathrm{C} = \mathrm{EI}\beta,$$

and, by substitution,

$$\mathrm{EI}\frac{d y}{d x} = \mathrm{EI}\beta + \mathrm{M}_1 x - \mathrm{S}_1\frac{x^2}{2} - \frac{w x^3}{6}.$$

Integrating again,

$$\mathrm{EI}y = \mathrm{EI}\beta x + \mathrm{M}_1\frac{x^2}{2} - \mathrm{S}_1\frac{x^3}{6} - \frac{w x^4}{24}.$$

The constant is omitted in this case because $y = 0$ when $x = 0$. Again, since $y = 0$ when $x = b$, we have—

$$0 = \mathrm{EI}\beta + \mathrm{M}_1\frac{b}{2} - \mathrm{S}_1\frac{b^2}{6} - \frac{w b^3}{24}. \quad . \quad . \quad (95)$$

Now, at a point Q, equally indefinitely near to but on the opposite side of B, we shall find the bending moment identical in value with that at the point P. We can therefore write a similar equation in this case, noting that a has a negative value. Thus—

$$0 = \mathrm{EI}\beta - \mathrm{M}_2\frac{a}{2} - \mathrm{S}_2\frac{a^2}{6} + \frac{w a^3}{24}.$$

Subtract, and for both M_1 and M_2 write M_B or the bending moment at the point B, to which they both approximate so closely as to be practically identical with it and each other. Accordingly,

$$\mathrm{M}_\mathrm{B}\left(\frac{a + b}{2}\right) + \mathrm{S}_2\frac{a^2}{6} - \mathrm{S}_1\frac{b^2}{6} - \frac{w}{24}(a^3 + b^3) = 0. \quad . \quad . \quad (96)$$

Again, taking moments about A for the portion A B—

$$\mathrm{M}_\mathrm{A} = \mathrm{M}_\mathrm{B} + \mathrm{S}_2 a - \frac{w}{2}a^2$$

and, similarly, about C for the portion B C—

$$\mathrm{M}_\mathrm{C} = \mathrm{M}_\mathrm{B} - \mathrm{S}_1 b - \frac{w}{2}b^2.$$

Multiply the first equation throughout by $-a$, the second by b, and re-arrange—

$$S_2 a^2 = -M_B a + M_A a + \frac{w}{2} a^3$$

$$S_1 b^2 = M_B b - M_C b - \frac{w}{2} b^3.$$

Subtract

$$S_2 a^2 - S_1 b^2 = -M_B (a + b) + M_A a + M_C b + \frac{w}{2} (a^3 + b^3).$$

Divide by 6, and substitute in equation (96) above—

$$M_B \frac{a+b}{2} - M_B \frac{a+b}{6} + M_A \frac{a}{6} + M_C \frac{b}{6} + \frac{w}{12}(a^3 + b^3) - \frac{w}{24}(a^3 + b^3) = 0,$$

which reduces to

$$M_A a + M_C b + 2 M_B (a + b) + \frac{w}{4} (a^3 + b^3) = 0. \qquad (97)$$

This equation is known as the *Theorem of Three Moments*, and its first enunciation is attributed to Clapeyron. By means of the relationship thus established, if the bending moments at two of the points of support of a uniformly loaded beam are known, the third can be deduced. The bending moments at the end supports are sufficiently obvious. If the beam project a distance, c, beyond the outer support, C, the moment at C is $\frac{w c^2}{2}$. If the beam do not project, the moment at the point of support is zero.

The shearing stresses can then be obtained from the formula already given, viz. :—

$$S_1 = \frac{M_B}{b} - \frac{M_C}{b} - \frac{w b}{2}$$

$$-S_2 = \frac{M_B}{a} - \frac{M_A}{a} - \frac{w a}{2}.$$

The shear at any point, X, is $S_1 - w x$. Accordingly, at A and C it is $S_A = -S_2 + w a$ and $S_C = S_1 - w b$ respectively.

From these, the reactions at the points of support are readily forthcoming, for $R_A = S_A$ and $R_C = S_C$, if there be no overhang. If there be an overhanging portion, as c at C, $R_C = S_C + w c$.

Also

$$R_B = S_2 - S_1 = \frac{M_A}{a} + \frac{M_C}{b} - M_B \left(\frac{a+b}{a b}\right) + \frac{w}{2}(a + b). \qquad (98)$$

Assuming that there is no overhang this equation simplifies into

$$R_B = \frac{w}{8}(a + b) \left\{\frac{a^2 + 3 a b + b^2}{a b}\right\}.$$

Equation (98) may be confirmed by an independent investigation which is worthy of notice, for it gives an expression for the current moment in terms of the moments at the points of support.

If A B (fig. 401) be a portion of a weightless beam between any two supports, P Q, with bending moments, y_1 and y_2, at A and B respectively, due to some external system of loading, it is clear that the line of moments between A and B will be right, and by a simple application of geometrical principles

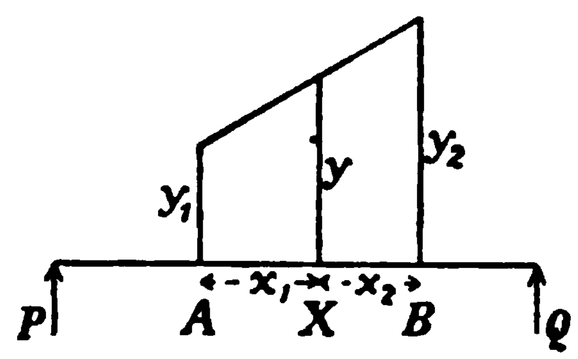


Fig. 401.

$$y(x_1 + x_2) = y_1 x_2 + y_2 x_1. \quad . \quad . \quad (99)$$

If, however, the beam be not weightless, or be loaded with a weight of w lbs. per foot run, the curve of moments is parabolic and the equation becomes—

$$y(x_1 + x_2) = y_1 x_2 + y_2 x_1 + \frac{w}{2} x_1 x_2 (x_1 + x_2). \quad . \quad . \quad (100)$$

The foregoing relationship is, of course, conditional upon there being no point of support between A and B. When such is not the case, and there is an upward reaction, R, at the point, X, we must expand the expression still further into

$$y(x_1 + x_2) = y_1 x_2 + y_2 x_1 + \frac{w}{2} x_1 x_2 (x_1 + x_2) - R x_1 x_2. \quad (101)$$

Re-arrange, and divide throughout by $x_1 x_2$,

$$R = \frac{y_1}{x_1} + \frac{y_2}{x_2} - y\left(\frac{x_1 + x_2}{x_1 x_2}\right) + \frac{w}{2}(x_1 + x_2), \quad . \quad (102)$$

an equation which is identical with the value of R_B given above, when the notation has been adapted thereto.

The second equation (100) in the preceding group yields us an expression for the current bending moment at any point, X, intermediate between the points of support.

$$M_X b = M_B x + M_C (b - x) + \frac{w b x}{2}(b - x). \quad . \quad (103)$$

If we revert to the case in which there are no moments at the end supports, we may derive the amounts of reaction at these points very readily, as follows:—

From equation (97) we have

$$2 M_B (a + b) = -\frac{w}{4}(a^3 + b^3),$$

or.

$$-M_B = \frac{w}{8}(a^2 - a b + b^2). \quad . \quad . \quad . \quad (104)$$

Also, from a consideration of the conditions of equilibrium to the left of B,

$$M_B = R_A a - \frac{w a}{2}.$$

Combining,

$$R_A a = -\frac{w}{8}(a^2 - a b + b^2) + \frac{w a^2}{2},$$

$$R_A = \frac{w}{8 a}(3 a^2 + a b - b^2),$$

and a similar expression may be written for R_C.

In the preceding investigation, for the sake of simplicity, it has been assumed that the three points of support are on a level. If this is not so, and the supports, A and C, are respectively heights of y_1 and y_2 above B (the heights being small), it is not difficult to establish, in the same manner, that

$$M_A a + M_C b + 2 M_B (a + b) + \frac{w}{4}(a^3 + b^3) = 6 E I \left(\frac{y_1}{a} + \frac{y_2}{b}\right). \quad (105)$$

And it is also clear that, if the lengths a and b be subjected to different loads, as w_1 and w_2 per foot run respectively, the equation will then become

$$M_A a + M_C b + 2 M_B (a + b) + \frac{w_1 a^3}{4} + \frac{w_2 b^3}{4} = 6 E I \left(\frac{y_1}{a} + \frac{y_2}{b}\right).$$

It would take too long, and it is unnecessary, to elaborate the formulæ for these cases in detail. The preceding method may be followed, and it will be found that where a level girder, without overhangs, is subjected to different intensities of load upon its two sections, the reactions are given by

$$R_A a (a + b) = w_1 a^2 \left(\frac{3 a}{8} + \frac{b}{2}\right) - w_2 \frac{b^3}{8}. \quad . \quad . \quad (106)$$

$$R_C b (a + b) = w_2 b^2 \left(\frac{3 b}{8} + \frac{a}{2}\right) - w_1 \frac{a^3}{8}. \quad . \quad . \quad (107)$$

$$R_B = w_1 a + w_2 b - (R_A + R_C). \quad . \quad . \quad (108)$$

We now come to the question of counterpoise. No notice has hitherto been taken of the effect exercised by the ballast at the tail end of the bridge, because it is much more convenient to consider this question separately from that of the uniform load of the structure generally, and afterwards to combine the results obtained in the two investigations.

To arrive at the stresses due to a sectional load, we must first consider those due to a concentrated load. As before, let A B C (fig. 402) be a

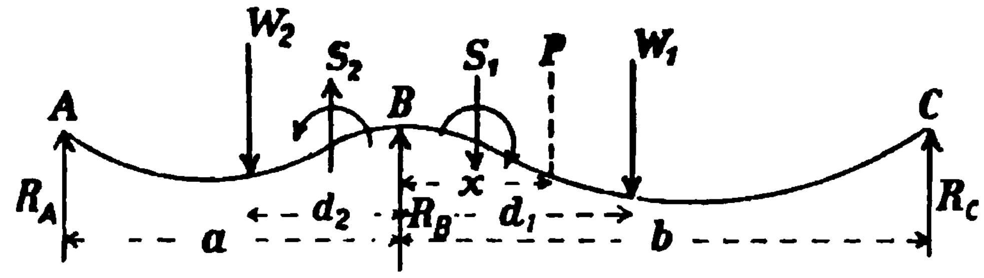


Fig. 402.

girder continuous over three points of support, A, B, and C, and let W_1 and W_2 be concentrated loads at distances, d_1 and d_2, from the central support.

Take any point, P, between B and W, at a distance, x, from B, the origin of co-ordinates.

Then, as already established, for the equilibrium of the portion B P,

$$EI\frac{d^2y}{dx^2} = M_B - S_1 x.$$

Integrate

$$EI\frac{dy}{dx} = C + M_B x - S_1\frac{x^2}{2},$$

and as $\frac{dy}{dx} = \tan\beta$ when x is indefinitely small, so in the limit,

$$C = EI\beta,$$

and

$$EI\frac{dy}{dx} = EI\beta + M_B x - S_1\frac{x^2}{2}. \qquad (109)$$

Integrating again,

$$EIy = EI\beta x + M_B\frac{x^2}{2} - S_1\frac{x^3}{6}. \qquad (110)$$

There is no constant, since x and y vanish together.

These equations hold good for all values of x between $x = 0$ and $x = d_1$.

Next, let the point, P, lie between W_1 and C, and remove the origin of co-ordinates to C. Then

$$EI\frac{d^2y}{dx^2} = -S_C x.$$

Integrating and determining the constant as before,

$$EI\frac{dy}{dx} = EI\alpha - S_C\frac{x^2}{2}, \qquad (111)$$

and again,

$$EIy = EI\alpha x - S_C\frac{x^3}{6}. \qquad (112)$$

Now, these two pairs of equations, though possessing different co-ordinates, have two conditions in common, viz.:—

(1) At the point, W_1, the value for y must be the same in each case.

(2) At the same point the slope or gradient is the same, but measured in opposite directions—*i.e.*,

$$\left(\frac{dy}{dx}\right)_1 + \left(\frac{dy}{dx}\right)_2 = 0.$$

Hence, substituting d_1 for x in equation (110) and $(b - d_1)$ for x in equation (112), we deduce from the first condition,

$$EI\beta d_1 + M_B\frac{d_1^2}{2} - S_1\frac{d_1^3}{6} = EI\alpha(b - d_1) - S_C\frac{(b - d_1)^3}{6}. \qquad (113)$$

Also, substituting likewise in equations (109) and (111), and using the second relation,

$$EI\beta + M_B d_1 - S_1\frac{d_1^2}{2} + EI\alpha - S_C\frac{(b - d_1)^2}{2} = 0. \qquad (114)$$

Next, let us consider the span, b, as a whole, and take moments about the points B and C respectively. In the first case, we have

$$S_0 b - W d_1 + M_B = 0, \quad \ldots \quad (115)$$

and in the second,

$$S_1 b - W(b - d_1) - M_B = 0. \quad \ldots \quad (116)$$

Substitute the values for S_1 and S_0 given by these equations in (113) and (114), re-arranging as below—

$$E I [\beta d_1 - \alpha (b - d_1)] = M_B \frac{b}{6}(b - 3 d_1) - W \frac{d_1}{6}(b - d_1)(b - 2 d_1).$$

$$E I (\beta + \alpha) = - M_B \frac{b}{2} + W \frac{d_1}{2}(b - d_1). \quad . \quad (117)$$

Multiply the latter equation by $(b - d_1)$ and eliminate α by addition—

$$E I \beta b = - M_B \frac{b^2}{3} + W \frac{d_1}{6}(b - d_1)(2b - d_1). \quad . \quad (118)$$

Now, let us deal in a similar manner with the span, a, to the left of the point, B. It is only necessary to re-write the previous equation, making the requisite changes in sign—

$$- E I \beta a = - M_B \frac{a^2}{3} + W \frac{d_2}{6}(a - d_2)(2a - d_2). \quad (119)$$

Whence, eliminating β between the two equations, we get

$$M_B = \frac{1}{2(a+b)} \left\{ \frac{W_1 d_1 (b - d_1)(2b - d_1)}{b} + \frac{W_2 d_2 (a - d_2)(2a - d_2)}{a} \right\}, \quad (120)$$

an expression which furnishes us with the value of the bending moment at the intermediate support, B. The bending moment at each end is, of course, zero.

It is sufficiently obvious that, when there is a load on only one of the spans (as W_1 on the span, b) the bending moment at the intermediate support is given by

$$M_B = \frac{1}{2(a+b)} \left\{ W_1 \frac{d_1 (b - d_1)(2b - d_1)}{b} \right\}, \quad \ldots \quad (121)$$

and that, for any system of concentrated loads on a single span, the general equation may be written—

$$M_B = \frac{1}{2b(a+b)} \Sigma \{ W d (b - d)(2b - d) \}. \quad \ldots \quad (122)$$

The reactions at the point of supports will be easily determined from a consideration of the conditions of equilibrium in each span. Assuming one span to be loaded as above—

$$M_B = - R_A a. \quad \ldots \quad (123)$$

$$M_B = R_C b - \Sigma W d. \quad \ldots \quad (124)$$

Substituting these values for M_B in (122), we find—

$$R_A = \frac{1}{2ab(a+b)} \Sigma \{W d (b-d)(2b-d)\}. \quad . \quad . \quad . \quad (125)$$

$$R_C = \frac{1}{2b^2(a+b)} \Sigma \{W d (b-d)(2b-d)\} + \frac{1}{b} \Sigma W d. \quad . \quad . \quad (126)$$

$$R_B = \Sigma W - (R_A + R_C). \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (127)$$

Having obtained the equations corresponding to a concentrated load, we now proceed a step further to obtain an expression for the bending moment due to a uniform load, w, of length, l, less than one span, b (see fig. 403).

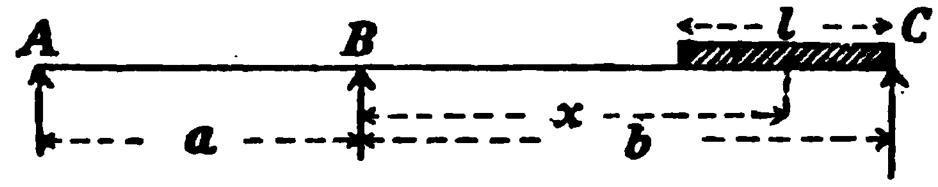


Fig. 403.

The weight on an infinitesimal length, dx, at a distance, x, from B is $w \,.\, dx$, and, taking the sum between the limits, $x = (b - l)$ and $x = b$,

$$M_B = \int_{b-l}^{b} w \,.\, dx \,.\, \frac{x(b-x)(2b-x)}{2b(a+b)} = w l^2 \frac{2b^2 - l^2}{8b(a+b)}. \quad . \quad . \quad (128)$$

The reactions at the points of support will therefore be—

$$R_A = -\, w l^2 \frac{2b^2 - l^2}{8ab(a+b)}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (129)$$

$$R_C = w l \left\{ 1 - \frac{l(6b^2 + 4ab - l^2)}{8b^2(a+b)} \right\}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (130)$$

$$R_B = w l - (R_A + R_C). \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (131)$$

These results afford us all the data for dealing not only with the stresses due to the counterpoise, but also with those due to a moving load covering the span to any desired extent. Having taken in detail the dead load, the counterpoise, and the moving load, it is only necessary to compute their algebraical sum in order to find the stresses due to the combined systems.

It would be possible to pursue the subject much further, but we have now reached the boundary which divides movable bridges from stationary bridges. The remaining calculations are common to both forms of structure, and the student is accordingly referred, for further information, to treatises dealing with the latter subject, in a more complete and efficient manner than is possible within the limits of the present volume.

Loads imposed on Movable Bridges.—Before designing the framework of a movable bridge, and in order that the stresses in the proposed members may be calculated, an estimate has to be made of the loads which the bridge will be called upon to bear. These loads may be classified as follows :—

1. *Dead Load.* — (α) Weight of main girders and bracing.
   (β) Weight of roadway or railway.
2. *Moving Load.*—(γ) Weight of trains, vehicles, &c.

Dead Load.—The weight of the main framework can be calculated in detail from the following data:—The weights of a square foot of cast iron, wrought iron, and steel, 1 inch in thickness, are 37·5, 40, and 40·8 lbs. respectively. But the process would necessitate a design too detailed for merely tentative purposes, and the calculations would be too lengthy for a preliminary estimate. A sufficiently accurate approximation, for practical purposes, can be obtained by the use of some empirical formula, based on existing examples. Trautwine * gives the following:—

For lengths not exceeding 75 feet, the weight in lbs. per foot run of two trusses or main girders, with lateral bracing for a single track,

$$W = \cdot 5 \times \text{span in feet} + 50 \sqrt{\text{span in feet}}.$$

For spans between 75 and 250 feet,

$$W = 4\cdot 5 \times \text{span in feet} + 22 \sqrt{\text{span in feet}}.$$

For double-track bridges, add 80 per cent. to the above values, and for narrow-gauge tracks, take 75 per cent. of the standard (4 feet 8½ inches) gauge.

The foregoing formulæ do not include any provision for the weight of cross girders, flooring, or rails.

The weight per foot run of iron floor systems, comprising a longitudinal stringer under each rail, is given by the same authority, as follows:—

Span.	Single Track.	Double Track.
20 to 100 feet.	200 to 275 lbs.	550 to 700 lbs.
100 „ 250 „	250 „ 350 „	700 „ 800 „

Exclusive of the main girders of a bridge, the dead load, consisting of iron or timber flooring slightly covered with ballast, the permanent way, cross girders with gusset attachments to main girders, and the horizontal bracing, of a double line of railway carried upon two main girders, may be estimated, according to Sir Benjamin Baker, as follows:—

10 to 100 feet span, ...	14 cwts.
100 „ 150 „ „	15 „
150 „ 200 „ „	16 „

Where the two lines of railway are supported upon three main girders, the above loads may be reduced by 2 cwts. per foot, and where upon four girders, by 4 cwts. per foot.†

"The weight of the cross girders and bracing for a railway bridge, to carry two lines of railway between main girders, may be taken on an average to vary from 6·7 cwts. for a 20-foot span to 9 cwts. for a 275-foot span; but it will be understood that considerable modification in these weights, both of a plus and minus nature, may be effected by a variation in the depth or arrangement of the cross girders."‡

* *The Civil Engineer's Pocket Book*, 17th ed., p. 604.

† Baker on "Short Span Railway Bridges." ‡ *Ibid.*

The following tabular values for the weight, in cwts. per foot run, of the main girders and of the entire bridge are condensed from an extensive compilation of data, from existing railway bridges, by Sir Benjamin Baker.*

TABLE XXXII.

Span in Feet.	Plate Girders.						Lattice Girders.			
	Two Main Girders with Lower Cross Girders.		Two Main Girders with Upper Cross Girders.		Four Main Girders under Rails without Cross Girders.		Two Main Girders with Lower Cross Girders.		Three Main Girders with Lower Cross Girders.	
	Main Girders.	Total Iron-work.	Main Girders.	Total Iron-work.	Main Girders.	Total Iron-work.	Main Girders.	Total Iron-work.	Main Girders.	Total Iron-work.
	Cwts.	Cwts.	Cwts.	Cwts.	Cwts.	Cwts.	Cwts.	Cwts.	Cwts.	Cwts.
20	3·4	10·1	3·4	8·2	4·7	6·2	3·1	9·8	3·7	8·3
25	3·9	10·6	3·9	8·7	5·5	7·1	3·6	10·3	4·2	8·9
30	4·4	11·1	4·4	9·2	6·2	7·8	4·1	10·8	4·7	9·4
35	5·0	11·8	5·0	9·9	7·0	8·6	4·5	11·3	5·3	10·0
40	5·5	12·3	5·5	10·4	7·8	9·5	5·0	11·8	5·8	10·6
45	6·1	12·9	6·1	11·0	8·6	10·3	5·4	12·2	6·3	11·1
50	6·6	13·5	6·6	11·6	9·3	11·1	5·9	12·8	6·9	11·8
60	7·7	14·7	7·7	12·8	11·0	12·8	6·8	13·8	7·9	12·8
70	8·8	15·9	8·8	14·0	12·4	14·3	7·7	14·8	9·0	14·0
80	9·9	17·2	9·9	15·1	14·1	16·0	8·6	15·8	10·1	15·1
90	11·0	18·3	11·0	16·3	15·8	17·8	9·5	16·8	11·1	16·2
100	12·1	19·5	12·1	17·5	17·7	19·8	10·4	17·8	12·2	17·4
120	14·3	21·9	14·3	19·8	...	...	12·2	19·8	...	...
140	16·5	24·3	16·5	22·1	...	...	14·0	21·8	...	...
160	18·7	26·7	18·7	24·4	...	...	15·9	23·9	...	...
180	20·9	29·1	20·9	26·8	...	...	17·7	25·9	...	...
200	23·1	31·5	23·1	29·1	...	...	19·5	27·9	...	...

The preceding formulæ and data are only relatively applicable to movable bridges under certain modifications and restrictions. The structure of a swing bridge has necessarily to be more substantial than that of a fixed bridge for the same span, on account of the more exacting nature of its functions and also because it has to be provided with a heavy pivot girder with other fittings. From a comparison of the lengths and weights of a number of existing bridges, the writer has found the total structural weight per foot run (exclusive of counterpoise) to range, generally speaking, from about 1 ton for small spans to 2 tons for large spans; there being, of course, instances in which these limits are not maintained. For moderate spans, say openings of from 50 to 125 feet, the dead load might fairly be estimated at 30 cwts. per foot run of the extreme length of the bridge, allowing for the accommodation of a double railroad track and a double footway.

Live Load.—While in a long bridge the weight of the locomotive and its tender may form a comparatively small proportion of the loaded length

* Baker on "Short Span Railway Bridges."

due to a long train, in a swing bridge, the span of which is small, the contingency of a continuous line of engines upon the bridge should be provided for.

The following table gives a statement of the weight of some recent locomotives and their tenders :—

TABLE XXXIII.—WEIGHT OF MODERN LOCOMOTIVES.*

Name of Railway.		Length Over All.	Wheel Base.	Total Weight.	Weight per Foot Run Over All.	Weight per Foot Run of Wheel Base.
		Feet.	Feet.	Tons.	Tons.	Tons.
Midland,[1]	Passenger engine,	53·2	44·1	85·48	1·61	1·95
	Goods engine,	49·7	37·0	76·46	1·54	2·03
	Tank engine,	33·4	22·0	51·02	1·58	2·32
London and North-Western,[1]	Passenger engine,	51·7	44·0	80·50	1·56	1·83
	Goods engine,	51·8	39·8	75·85	1·46	1·91
	Tank engine,	33·7	22·4	52·30	1·56	2·35
Lancashire and Yorkshire,[1]	Passenger engine,	50·0	40·9	70·92	1·40	1·72
	Goods engine,	48·7	36·0	68·25	1·40	1·90
	Tank engine,	38·7	24·3	59·15	1·53	2·43
North-Eastern,[1]	Passenger engine,	56·5	46·5	91·90	1·62	1·97
	Goods engine,	50·1	47·9	75·65	1·51	2·00
	Tank engine,	35·9	22·5	55·22	1·54	2·45
Great Western,[1]	Passenger engine,	57·5	47·5	84·65	1·47	1·78
	Goods engine,	57·9	48·9	93·00	1·61	1·91
	Tank engine,	30·4	15·5	47·00	1·55	3·03
Belgian State,[1]	Passenger engine,	57·3	49·0	104·21	1·82	2·12
Prussian State,[4]	Goods engine,	32·6	...	56·40	1·73	...
Swedish State,[5]	Tank engine,	...	13·0	57·00	...	4·40
Union Pacific, U.S A.,[2]	Passenger engine,	...	26·0	82·25	...	3·16
Chicago and North-Western, U.S.A.,[2]	...	...	27·0	71 50	...	2·65
Pennsylvania, ,, [3]	Passenger engine and tender,	...	47·5	77·25	...	1·63

From the preceding table it will be seen that the ordinary concentrated rolling load incurred by bridges in the United Kingdom at the present time may be taken at from 30 to 35 cwts. per foot run for each line of rails. In view of the likelihood of heavier developments, however, in the future, 2 tons or even 2¼ tons would be by no means an excessive allowance. Even these figures are exceeded in certain cases, as is evident from a table † prepared by Mr. Alexander Ross, the engineer to the Great Northern Railway Co., of which an abridgement is given below. The table shows the equivalent uniformly distributed live loads derived from the maximum

* NOTE.—These statistics are derived from the following sources :—

[1] Fair on "Moving Loads on Railway Underbridges," *Min. Proc. Inst. C.E.*, vol. cxli.

[2] Leigh on "American Passenger Locomotives," *Min. Proc. Inst. C.E.*, vol. cxlvi.

[3] Trautwine's *Civil Engineers' Pocket-Book.*

[4] Von Borries on "Prussian State Railways," *Min. Proc. Inst. C.E.*, vol. cxxii.

[5] *Teknish Tedskrift*, Stockholm, Oct. 31, 1901, and *Min. Proc. Inst. C.E.*, vol. cxliv., p. 340.

† Ross, on "Railway Bridges," Eng. Conf., 1903, *vide Engineering*, June 19, 1903.

bending moment caused by representative heavy engines running on British railways, with an addition of $2\frac{1}{2}$ per cent. for possible future increase.

TABLE XXXIV.—EQUIVALENT DISTRIBUTED LIVE LOADS DERIVED FROM MAXIMUM BENDING MOMENTS FOR A SINGLE LINE OF WAY.

Span in Feet.	Selected Engines.									
	Single Driver.		4-Wheel Coupled.		6-Wheel Coupled.		8-Wheel Coupled.		10-Wheel Coupled.	
	Tons Distributed.	Tons per Ft. Run.	Tons Distributed.	Tons per Ft. Run.	Tons Distributed.	Tons per Ft. Run.	Tons Distributed.	Tons per Ft. Run.	Tons Distributed.	Tons per Ft. Run.
10	36·9	3·69	36·9	3·69	36·9	3·69	34·6	3·46	39·9	3·99
15	38·1	2·54	46·6	3·11	48·8	3·25	50·28	3·35	55·8	3·72
20	44·0	2·20	57·6	2·88	56·2	2·81	63·5	3·18	68·9	3·44
25	51·5	2·06	65·4	2·61	66·3	2·65	73·8	2·95	83·7	3·35
30	61·2	2·04	73·6	2·45	74·7	2·49	83·2	2·77	98·5	3·28
35	71·1	2·03	82·6	2·36	84·0	2·40	91·4	2·61	106·9	3·06
40	80·4	2·01	89·0	2·22	92·4	2·31	98·8	2·47	115·3	2·88
45	90·0	2·00	95·6	2·12	99·0	2·20	105·6	2·34	120·2	2·67
50	99·0	1·98	105·3	2·10	104·0	2·08	112·3	2·24	125·0	2·50
60	116·0	1·93	124·8	2·08	117·6	1·96	126·0	2·10	136·3	2·27
70	135·3	1·93	142·8	2·04	132·3	1·89	140·5	2·01	158·9	2·27
80	152·7	1·91	160·4	2·00	150·4	1·88	159·2	1·99	180·8	2·26
90	172·0	1·91	176·4	1·96	168·3	1·87	176·4	1·96	202·5	2·25
100	188·6	1·88	193·3	1·93	186·0	1·86	192·0	1·92	223·7	2·24
125	233·8	1·87	240·0	1·92	232·5	1·86	240·0	1·92	278·7	2·23
150	277·5	1·85	288·0	1·92	279·0	1·86	286·5	1·91	333·0	2·22
175	316·8	1·81	336·0	1·92	325·5	1·86	332·5	1·90	385·0	2·20
200	352·3	1·76	383·2	1·92	370·0	1·85	378·0	1·89	435·3	2·18

For cartways and vehicular tracks, a rolling load of 10 to 15 cwts. per foot run should be a sufficient estimate in ordinary cases. Special vehicles may, however, carry loads equivalent to a ton per foot run. Floats or lorries for heavy goods vary in size between about 14 feet 9 inches by 6 feet 9 inches to 17 feet 6 inches by 7 feet 6 inches. The former generally carry loads up to 7 or 8 tons and the latter up to 10 or 11 tons, though 12, and even 15, tons may be reached under certain circumstances. Traction engines will exert a pressure of 300 to 600 lbs. per square foot over the area of their wheel-bases.

The weight of a crowd of men is generally taken at 80 to 84 lbs. per square foot. Dr. Stoney records an experiment in which he succeeded in packing a number of labourers in an enclosure, so closely as to produce a pressure of 147 lbs. per square foot. It would not be injudicious, therefore, to assume 100 lbs., or even 1 cwt., as the possible amount of concentrated load upon footways.

Practical Application.—By way of illustration of the theoretical methods involved in designing a movable bridge, an outline of the calculations for finding the reactions at the points of support in a specific instance is appended.

Fig. 404 is the skeleton diagram of a swing bridge over a passage 100 feet wide. P is the position of the pivot upon which the bridge turns, and A, B, and C are the blocks which support the bridge in the closed position. Their respective distances apart are shown in the figure.

It is, first of all, necessary to assume a value for the anticipated dead and live loads. Let us take the former at 30 cwts. and the latter at 70 cwts. per foot run.

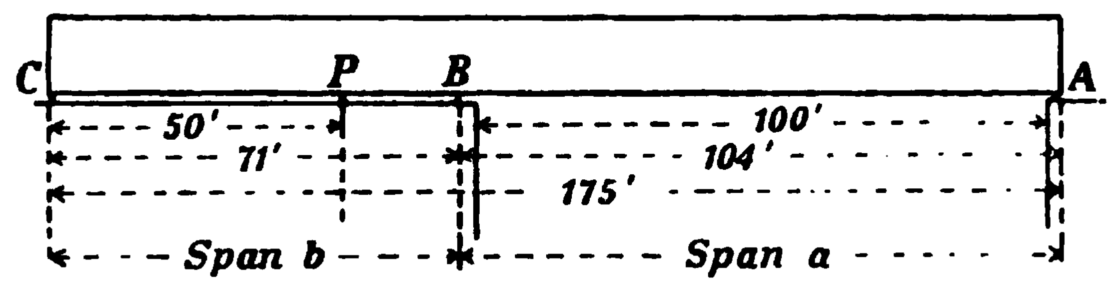


Fig. 404.

1. *To find the amount of ballast required.*—Suppose the ballast box to occupy a length of 16 feet at the tail-end of the bridge. Then the centre of gravity of the counterpoise will be 42 feet from the pivot, and, by taking moments about P,

$$42\ \mathrm{B} + 50 \times 1\tfrac{1}{2} \times \frac{50}{2} = 125 \times 1\tfrac{1}{2} \times \frac{125}{2}$$

$$\therefore\quad \mathrm{B} = 234 \text{ tons,}$$

where B is the quantity in tons of ballast required. To afford a margin of stability it will be as well to say 250 tons.

2. *To find the pivot reaction*—

Bridge structure, $175 \times 1\frac{1}{2}$, ...	$= 262\frac{1}{2}$ tons.
Ballast, ...	$= 250$ „
$\mathrm{R_P}$	$= 512\frac{1}{2}$ „

3. *To find the reactions of the bearing blocks.*—It will be convenient to consider the dead and live loads in combinations, adding the ballast later. This admits of the taking of four cases to cover the principal dispositions of the load :—

Case I.—Dead load throughout.
Case II.—Dead load on span a ; dead plus live load on span b.
Case III.—Dead plus live load on span a ; dead load on span b.
Case IV.—Live load throughout.

For the purpose in view it is only necessary to deal with one of these cases. Accordingly, we will select Case II. as typical of the group.

From formula (106)—

$$\mathrm{R_A}\, a\,(a+b) = w_1\, a^2 \left(\frac{3\,a}{8} + \frac{b}{2}\right) - w_2\,\frac{b^3}{8}$$

$$\mathrm{R_A}\,(104 \times 175) = 1\tfrac{1}{2} \times 104^2 \left(\frac{3 \times 104}{8} + \frac{71}{2}\right) - 5\,\frac{71^3}{8}$$

$$\mathrm{R_A} = 54{\cdot}1 \text{ tons.}$$

Similarly, from (107), $R_C = 142\cdot4$ tons,

and, by residue, $R_B = 314\cdot5$ „

Now take the ballast. From formulas (129), (130)—

$$R_A = - w l^2 \frac{2b^2 - l^2}{8ab(a+b)} = - 250 \times 16 \left(\frac{2 \times 71^2 - 16^2}{8 \times 71 \times 104 \times 175}\right)$$

$$= - 3\cdot7 \text{ tons};$$

$$R_C = w l \left\{1 - \frac{l(6b^2 + 4ab - l^2)}{8b^2(a+b)}\right\}$$

$$= 250 \left\{1 - \frac{16(6 \times 71^2 + 4 \times 71 \times 104 - 16^2)}{8 \times 71^2 \times 175}\right\} = 216 \text{ tons},$$

and, by residue,

$$R_B = 37\cdot7 \text{ tons}.$$

Hence, the nett reactions for the whole bridge under the conditions stated are—

$$\begin{aligned} R_A &= 54\cdot1 - 3\cdot7 = 50\cdot4 \text{ tons.} \\ R_B &= 314\cdot5 + 37\cdot7 = 352\cdot2 \text{ „} \\ R_C &= 142\cdot4 + 216\cdot0 = 358\cdot4 \text{ „} \\ & \qquad\qquad\qquad\quad 761\cdot0 \text{ „} \end{aligned}$$

The sum is the total weight of bridge structure, imposed load, and ballast.

Having determined the reactions at the points of support by calculation as above, it will be found most convenient to obtain the bending moment and shearing stress throughout the bridge by graphical methods. The diagrams admit of superposition, from which the points of maximum stress may be determined under any variation of loading. At this stage, however, the procedure is common to bridge design generally and need not be further investigated.

Distinctive Features of Movable Bridges.—The following essential and distinctive features of swing bridges claim some brief attention :—

The Pivot.—There are two main systems, or methods, in which a swing bridge is united with the pivot upon which it revolves—viz., the method of suspension and the method of superposition. In the latter instance, the body of the bridge rests directly upon the pivot in a manner analogous to the ordinary balancing of a bar upon any vertical. In the former system, the bridge structure is suspended from the pivot by means of stout bolts, which pass up from the underside of the pivot girders to the extremities of a crosshead, or saddle-piece, carried by the pivot.

The structure of the pivot itself follows an almost numberless variety of individual designs, dependent on one or other of the two principles adopted. We will accord a passing notice to a few typical cases.

(*a*) A long, narrow pivot passing through the bridge, nearly to the surface of the roadway, as at Velsen (fig. 405). Such a pivot requires a firm and unyielding foundation, for any inequality of settlement will

materially interfere with the working of the bridge. Further, owing to its slender proportions, it is very liable to fracture from shocks or impact due to abrupt stoppages and passing vessels. Accordingly, it must be well protected. The advantage of the design lies in the fact that a high pivot identifies the point of support more nearly with the centre of gravity of the bridge, or even places the support above it, and so conduces to steadiness

Fig. 405.—Bridge Pivot at Velsen.

of movement and absence of surging. This type of pivot can, of course, only be adopted when the plane of the roadway is some distance above the lower flanges of the bridge girders. Sometimes the conical form of the pivot is more accentuated, as in fig. 406, which is the pivot of a bridge at Rotterdam. A is the socket on which the pivot rests after passing through the cast-iron bearing girder, B.

(*b*) A short, stout pivot with a hemispherical head, as in fig. 407, taken from a bridge at Liverpool. This type naturally offers great resistance to

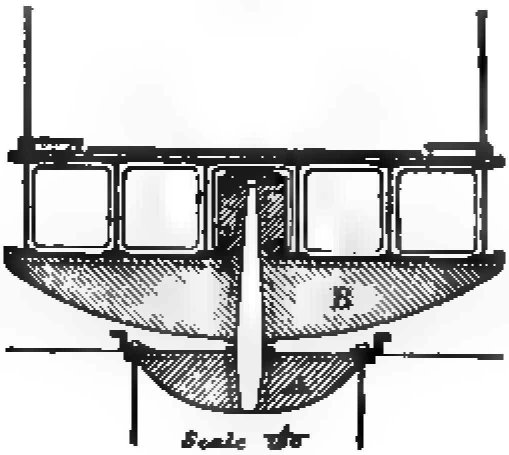


Fig. 406.—Bridge Pivot at Rotterdam.

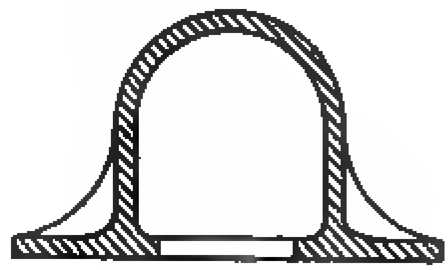


Fig. 407.—Bridge Pivot.

Fig. 408.—Raritan Bridge Pivot.

pressure and concussions, and affords a broad base for the distribution of stress. It allows of the bridge tilting slightly, but any decided movement in this direction may be checked by a ring of rollers, or by wheels at suitable points. In some instances a more pointed bearing surface will be found, as in fig. 408, showing the arrangement in the Raritan bridge. The rollers are here called upon to exercise considerable steadying effort.

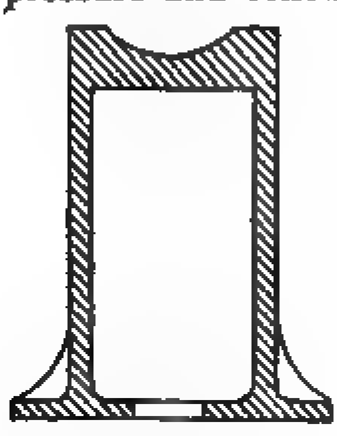
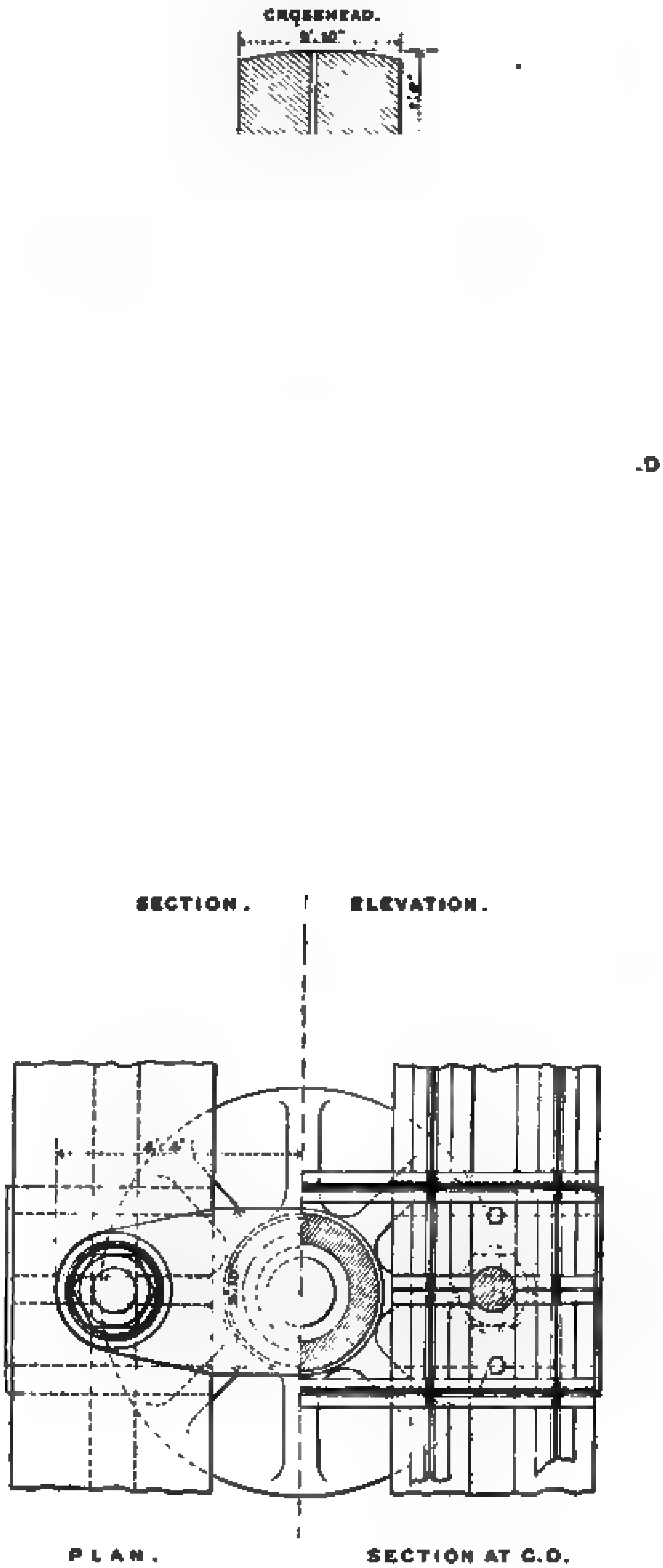


Fig. 409. Bridge Pivot.

(*c*) A long, cylindrical pivot with a concave seating or bearing—strictly speaking, a socket—as in fig. 409, exemplified in several forms. In one case, at Hawarden

(figs. 410 and 411), the method of suspension has been adopted, but in a kindred example at Liverpool (figs. 412 and 413) the bridge was superimposed. Tilting is very possible, and there is even a tendency to disturb the bridge to a dangerous extent in the absence of proper precautions. A bridge of this description was invaded one dinner hour, by a dense crowd of impatient working men before it had ceased swinging, with the result that it canted over forward, and a disaster was only averted by the nose-end coming into contact with, and resting upon, the passage gates. Consequent upon this mishap, the intermediate bearing blocks were made continuous throughout the arc of travel, so that excessive tilting at any stage of the rotation was rendered impossible on any future occasion.

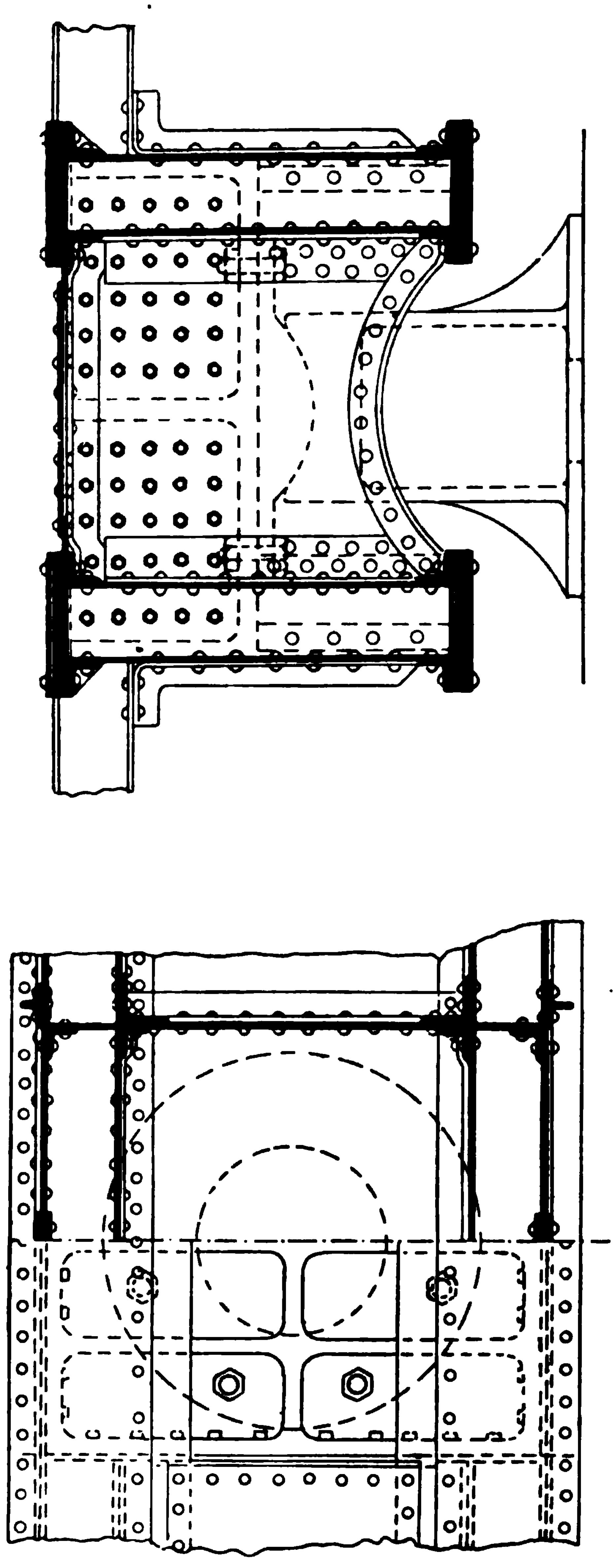


Figs. 410 and 411.—Bridge Pivot at Hawarden.

(*d*) A dwarf, cylindrical pivot, also with a concave seat, as at the Fleetwood bridge (figs. 414 and 415). Any overturning leverage exerted upon the support is reduced to a minimum, but the steadiness of the bridge is thereby lessened. A peculiar feature about the

pivot illustrated is that it is provided with keys or wedges, whereby the bridge can be more accurately balanced.

All the foregoing examples are instances of what may be termed the solid pivot, in contradistinction to the hydraulic pivot, exemplified in the two following cases.



Figs. 412 and 413.—Bridge Pivot at Liverpool—Scale, $\frac{3}{8}$ inch to 1 foot.

(*e*) A cylindrical pivot of medium height (figs. 416 and 417), with a perfectly plane top so disposed as to receive only the vertical pressure due to the weight of the bridge, the axis of the pivot passing through the centre of gravity of the bridge. Any lateral action due to surging or vibration is taken by a horizontal ring of small rollers encircling the lower part of the bridge seating. The pivot is essentially a ram or piston raised into position by hydraulic pressure against its under surface, and allowed to fall after the completion of the rotative work. This system is practised at Marseilles. With a slight modification it has been also practised at Liverpool. The modification consists in a concavity in the upper surface of the ram, to receive the hemispherical seating of the bridge, so that the latter may revolve about the pivot instead of the pivot turning in the cylinder with a tendency to wear the sides. As a matter of fact, it is difficult to ensure the immobility of the pivot, so that the object aimed at cannot be said to be achieved. It is important to note that there is a grave risk attaching to the apparently simple and effective contrivance just described. Should the hydraulic pressure not be

cut off, through any failure of the automatic apparatus, there is nothing to prevent the pivot being driven completely out of the cylinder, with disastrous consequences to the bridge. This has actually occurred in two instances to the author's knowledge. A solid pivot is, therefore, to be preferred on this account. Any accident to a bridge over an important waterway entails loss and inconvenience far exceeding the damage to the structure itself.

View transverse to the Bridge.

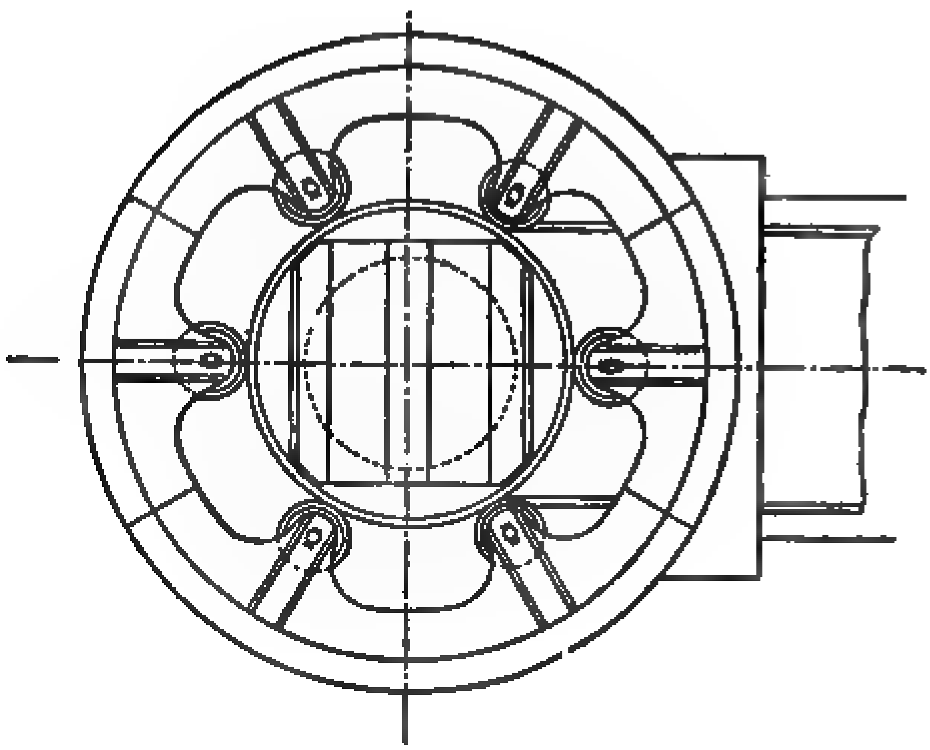
View longitudinal with the Bridge.

Figs. 414 and 415.—Bridge Pivot at Fleetwood.

(*f*) A water-borne carriage, consisting of a buoy continuously immersed, with a very small central pivot beneath it, taking about 5 per cent. only of the dead weight of the bridge. The carriage is steadied by a horizontal ring of wheels. Fig. 418 is an illustration of a pivot thus constructed at the Spencer Dock, Dublin.

A practical point worthy of notice is the very decided tendency exhibited

by swing bridges to wear their pivots unevenly. Owing to the pull exercised by the turning rams behind the pivot, the bridge bears more heavily against the forward side, and in process of time creeps gradually backward from its true centre, so as eventually to cause the tail of the bridge to jamb against the masonry of the bridge pit. This movement has been known to take place to the extent of an inch or more. A remedy might perhaps be found in a movable pivot provided with a base fitting into a fixed sole-plate, where it could be adjusted at intervals by means of cotters or wedges.

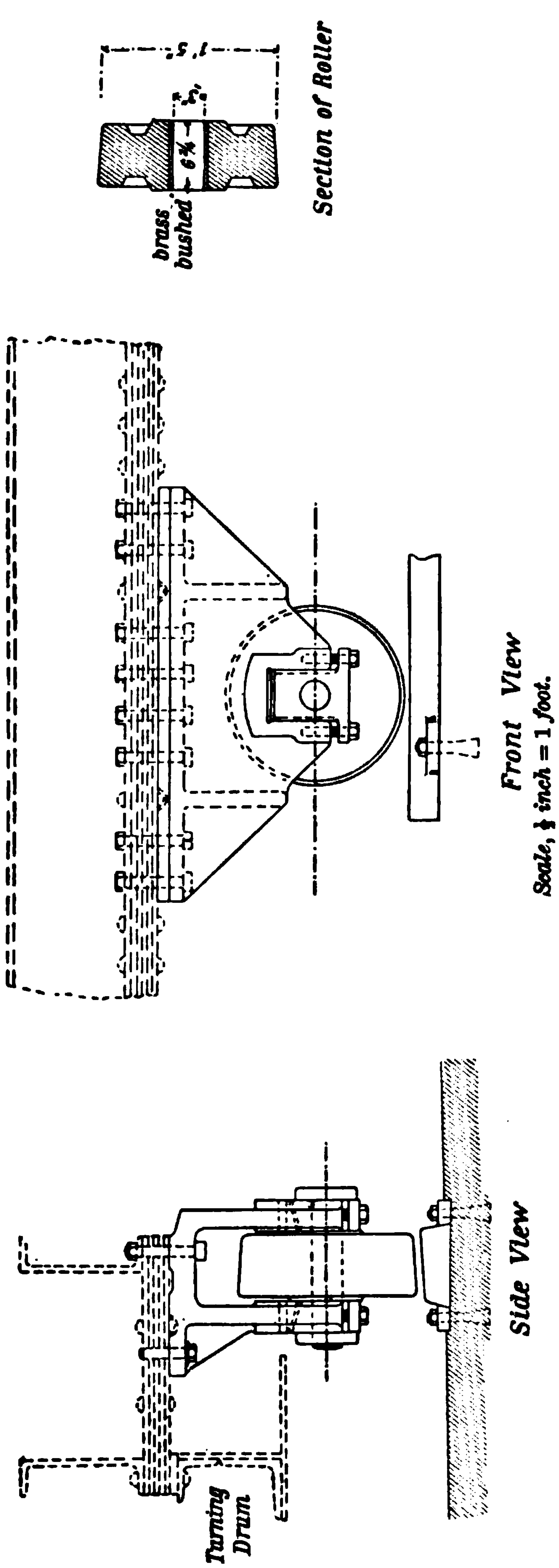


Figs. 416 and 417.—Bridge Pivot at Marseilles.

Balancing Rollers and Wheels.—For a swing bridge whose centre of gravity does not lie upon the axis of rotation, some additional supporting power is rendered necessary, and this is supplied by balancing wheels or rollers. In some cases (fig. 431) the weight is brought to bear upon the top

Fig. 418.—Swing Bridge at Dublin.

of a ring of rollers, which are either free to travel with the bridge or which simply revolve upon their own axes without progression. In other instances (figs. 419, 420, and 421) a series of two or more wheels is attached to the under side of the bridge and travel over a circular roller path. The weight is transmitted through the wheel axles, and the turning friction is considerably greater than with live rollers. Wheels do not always run upon the floor of the bridge pit. In some instances, the ballasting of the bridge is reduced to a minimum, and the centre of gravity lies forward of the pivot. The wheels are then placed at the extremity of the tail end and bear upwards against the under side of a corbel course or projection in the circumference of the bridge pit, which must necessarily be constructed in heavy blocks of masonry.



Figs. 419, 420, and 421.—Balancing Rollers and Roller Path.

For bridges accurately balanced over their centre of gravity, no additional support is required except for steadying purposes, and that only in the case of very light bridges, but it is nearly always provided, more perhaps as a precaution than as a necessity. The force required to disturb the stability of balanced heavy bridges is extremely great. M. Barret* alludes to a bridge at Marseilles which, with a length of 247 feet and a weight of 500 tons, would allow a two-wheel dray carrying 6 tons to mount one end of it at the moment of swinging without disturbing the longitudinal equilibrium, while a force of no less

* *Min. Proc. Inst. C.E.*, vol. lvii.

than 89 tons would have to be applied at its axis in order to affect the transverse equilibrium.

The great majority of swing bridges, however, have their weights distributed between the pivot and the wheels or rollers in varying proportions, capable of adjustment by mechanical contrivances. The revolving members must have conical surfaces with axes radiating to the centre of rotation. Their diameters in existing examples vary from about 8 inches to 5 feet; but such extremes are injudicious owing, in the first case, to the difficulty of obtaining a satisfactory adjustment and, in the second, to the great depth of the roller path. Between 18 inches and 3 feet will be found a suitable range for practical purposes. Large rollers, on account of the correspondingly obtuse angles which they subtend, have a tendency to work out of position under pressure. They are restrained by their inner flanges or by axial rods to the pivot, but in either case the friction is augmented.

Fig. 422.—Balancing Lever.

Sometimes a double wheel track is provided, or there is an intermediate row of friction rollers near the centre. In order to secure a proportionate pressure upon these intermediate supports, the bearing is communicated through a volute or other spring or by means of counter-weighting. This latter method is achieved by placing the wheel journals in a loose cast-iron frame connected with a balancing lever as shown in fig. 422.

Some bridges move entirely upon a turntable of rollers, leaving scarcely any appreciable weight to be borne by the pivot. A footbridge has been constructed which revolved upon a row of cannon balls between two grooved cast-iron plates.

The Counterpoise.—Masonry, gravel or rubble ballast, and cast-iron kentledge have all been utilised for the purpose of counterweighting movable bridges. The last-named material, being heavier and easy to mould in blocks of suitable shape and size, is most generally used, a very inferior quality of iron being employed.

The kentledge is deposited in a special compartment called the ballast box, arranged at the extremity of the tail end of the bridge, commonly below the floor level, though the space between the webs of box girders is also available for the purpose. The interior surfaces of the ballast box should be washed over with liquid Portland cement, and the interstices between the blocks run with grout, to prevent corrosion.

The counterpoise has occasionally been disposed as an ornamental feature, and a massive balustrade or an entrance arch in cast iron may be cited as illustrations. Such artistic pretensions are, however, in questionable taste in situations where the functions of a bridge are strictly utilitarian.

The amount of ballast required to give the requisite stability depends upon the ratio which the length of the tail bears to the length of the bridge forward of the pivot or point of support. A bridge with the pivot exactly at its centre, as is generally the case where two parallel openings have to be spanned, and also for some single openings, as at Naburn on the Ouse, near York, needs no counterweight. In the majority of cases a shorter tail is the rule for two reasons—first, on the ground of expense, for the structure of a bridge is far costlier than even a much greater dead weight of ballast; and secondly, there is less occupation of valuable quay space by a counterbalanced tail. In fact, at some sites, a short tail is absolutely unavoidable. The Whitehaven swing bridge has a tail only one-fourth of the total length, or one-third of the length of the forward portion. In a number of cases the proportion is one-half of the forward portion, while at Marseilles it is three-fifths. In an interesting paper on the subject, Mr. C. F. Findlay* demonstrates, by an application of the calculus, that if the cost per ton of the bridge structure be five times the cost of the kentledge, for any bridge not of extremely minute span, the length of tail should be approximately one-third of the length of the other section, if the most economical proportion is to be observed.

Bridges which depend for their stability upon the downward reaction of an inverted roller path do not of necessity require ballasting, if the path itself be secure.

Ballast being an unremunerative form of weight, an attempt has been made, in one case at least, to balance a bridge by placing the hydraulic rams, which work it, within the ballast box. This method necessitates a hollow pivot for the transmission of the water pressure. In the case of a small bridge, the paving of the short end with stone setts, and the long end with wood blocks, has been found an adequate solution of the difficulty.

Setting Apparatus.—For obvious reasons it is not advisable to allow a bridge to rest upon its pivot longer than is required for the operation of turning. While undergoing the stress due to moving loads, the structure is preferably supported on some independent base. Before the introduction of hydraulic power, when the usual practice was to carry the bulk of the

* Findlay on "The Design of Movable Bridges," *Min. Proc. L.E.S.*, vol. ii.

weight on a ring of live rollers, a single bridge was wedged up at each end until such time as it was necessary to put it in motion, when the wedges were withdrawn. A bridge with double leaves was also wedged up at the tail ends, so that each leaf tilted forward on to bearing blocks provided at the edge of the coping. The wedges were actuated by mechanical means, such as the screw and the lever.

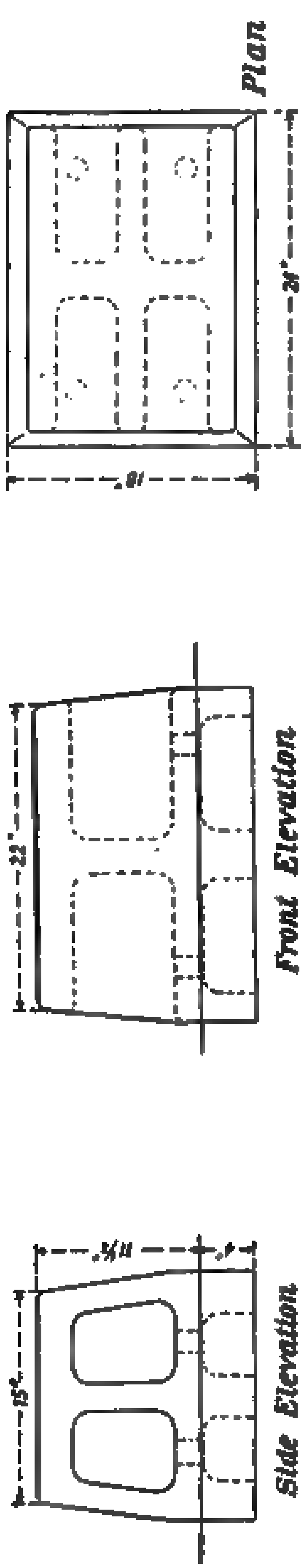
With the advent of hydraulic power came the water pivot, which raised the bridge off its fixed bearings during the process of rotation, and afterwards allowed it to return to them. The advantages of a solid pivot have caused the transference of the hydraulic lifting rams to the extreme rear, where the wedging-up process has been followed, but with this modification, that when the rams have lifted the bridge clear off the pivot, a pair of sliding bearing blocks are inserted, and the lifting power is withdrawn until it is required once more to raise the bridge for the removal of the bearing blocks and the resumption of the pivot seating (see fig. 441).

Another form of lifting apparatus is the knuckle or toggle gear, which consists essentially of two short bars linked together, and flexibly connected with an upper frame, constructed to move vertically, and a base which is fixed. When the two bars are in one vertical line, the upper plate is at its highest elevation, and any movement in the bars produces a depression in the level of the plate. The thrust of a hydraulic ram straightens the knuckle, so that a bearing block may be inserted as before, but in some cases the weight of the bridge continues to be borne by the gear, the links being driven slightly past the vertical position in order to preclude any tendency to a backward movement of the ram. The opposite motion is effected by another hydraulic cylinder. As a mechanical means, the toggle joint is very powerful. Eccentrics and cams on shafting and bent levers have also been employed to accomplish the necessary lift.

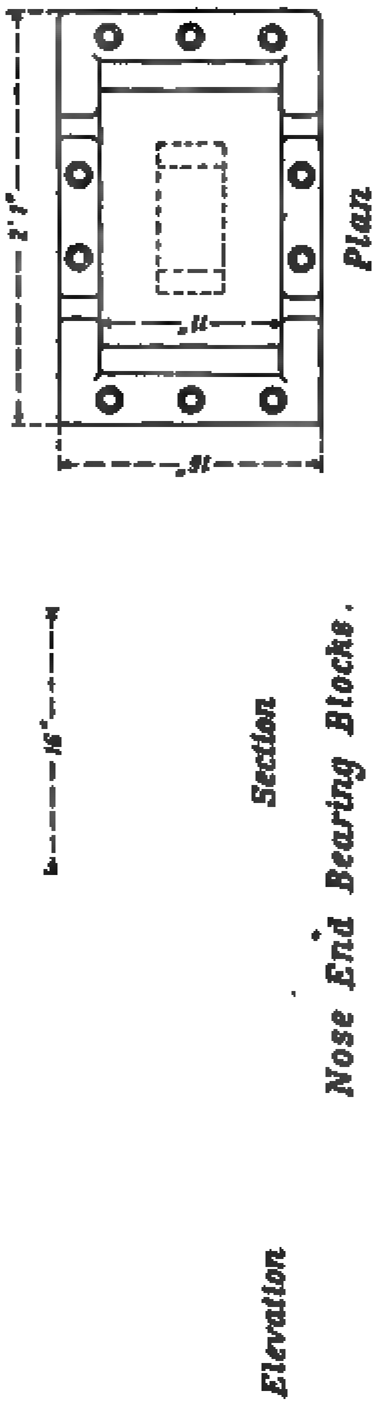
An ingenious arrangement adopted for a double-leaved swing bridge, each leaf weighing 116 tons, over an 80 feet passage at Barrow, consists in allowing the bridge to remain continuously upon the pivot through the medium of a very shallow and flexible girder. During the passage of a load over the bridge, this girder deflects sufficiently to admit of the structure coming in contact with specially arranged fixed blocks, which themselves take up the actual weight. After the transit of the load the resilience of the girder causes it to spring back to its original position and the bridge resumes the swinging condition. By this contrivance all apparatus for lifting and setting is dispensed with.

Examples of blocks provided for the centre bearing and the nose end of a bridge at Liverpool are given in figs. 423 to 428. The blocks for the tail end are similar to the centre bearing blocks, with the addition that their undersides, instead of being fixed, are arranged to slide in grooves in soleplates, as shown in fig. 441. The upper members in figs. 426 and 427 are attached to the underside of the bridge structure.

Interlocking Apparatus.—The two leaves of a double swing bridge are often locked together, not so much with the idea of forming a continuous structure, as for the purpose of equalising the deflection of the nose ends



Figs. 423, 424, and 425.—Centre Bearing Blocks.



Nose End Bearing Blocks.

Figs. 426, 427, and 428.

under a load approaching from one side only. Without some such arrangement there would be a perceptible difference in level between the

extremities of the loaded and unloaded leaves and a sharp recoil of one of them when the pressure had been transferred to the other.

This connection may take the form of a projection on one of the meeting faces with a corresponding groove in the other face, engagement being made in the ordinary process of rotation. Or, again, where the leaves tilt slightly after turning, so that a tongue-and-groove joint is not feasible, long bolts have been shot home through the faces of the leaves. The motive power in such cases may be a hand lever, a screw, or a hydraulic ram.

In a number of instances horizontal interlocking is omitted entirely, partly on account of the necessary clearance required for expansion and partly to avoid the inconveniences of a complicated adjustment. A simple plug dropped into a vertical dovetailed groove serves to unite the leaves and keep them in position.

General Notes on Design.

Having regard to the maximum resistance of the material to stress and the minimum thickness consistent with stiffness, one-ninth or one-tenth of the unsupported length will generally be found the most effective ratio for the depth of iron or steel girders at the point of support. Towards the nose end of the bridge a reduction is advisable, both on account of economy and headroom.

Except for very short spans, lattice girders are preferable to plate girders. The latter make a heavier bridge and expose a larger surface to wind pressure.

In bridges carrying a railway track, the cross girders must be designed to take at least the full concentrated load of a pair of engine driving wheels, say 16 to 19 tons, and in order that this intensity of pressure may not be exceeded, it is necessary that their distances apart should not be greater than the distance between two consecutive pairs of wheels, say 6 or 7 feet, while it cannot economically be much less than that amount. But in ordinary cases, 9 to 12 feet is considered a generally advantageous range, in which case the load on the cross girders is 32 tons for each track, exclusive of structural load. It will be found economical to give the cross girders a larger proportion of depth than the main girders, say one-seventh or one-eighth of their span. Wind bracing should be provided to withstand the authorised (but excessive) estimate of 56 lbs. per square foot.

An alternative to the cross girder system is to carry the rails on the main girders themselves, which accordingly must lie below the platform. This method, while diminishing the headroom of the closed passage, increases the effective breadth of the bridge by the flange width of two or more girders, which otherwise would protrude above the roadway level, and at the same time provides a clear deck, flush with the coping, when the bridge is swung back into its recess. On the other hand, a deeper

bridge pit is required and the system cannot be adopted in the case of low quays.

From an investigation made by Mr. Findlay, it appears that for a rolling load of 10 cwts. per foot per girder, a single-leaf swing bridge is more economical than a double-leaf bridge up to 150 feet span. If the rolling load be increased to 1 ton per foot run per girder, the economical limit of single-leaf bridges is raised to 180 feet.

Folding or Lowering Bridge at Greenock.*

This forms the superstructure to a caisson closing the entrance to the Garvel Graving Dock at Greenock, and already alluded to in Chap. viii. It is only necessary to supplement the account there given of the whole structure with some particulars relating exclusively to the bridge portion, which is the design of the late Mr. W. R. Kinipple (figs. 429 and 430).

The bridge roadway is carried by a series of parallel axles in pairs, placed vertical over one another, transversely to the bridge, at a distance of 30 inches, each pair being connected by four parallel rocking bars working freely on both axles. The outer bars are prolonged above the roadway level to form standards for handrailing. Two pairs of the inner bars are extended downwards into a watertight chamber of the caisson, where they are attached to boxes of ballast which act as counterweights. The raising or lowering of the bridge platform is effected by rollers fixed on each end, which work against curved plates in the abutment and the curved girder or lowering plate across the entrance to the recess. The process of hauling the caisson into its chamber brings the inner roller in contact with a convex plate, causing the handrail and platform automatically to fall to a lower level. The opening of the bridge consists of the reverse process. The outer rollers of the platform come in contact with a concave plate which causes the platform to rise to the quay level. When the bridge is in position it is locked between the abutments so that it cannot fall, and in such a way that it does not vibrate under the heaviest traffic. The plummer blocks carrying the rocking bars are 9 feet 9 inches apart, and are supported on plated columns extending to the bottom of the caisson.

Traversing Bridge at Antwerp.†

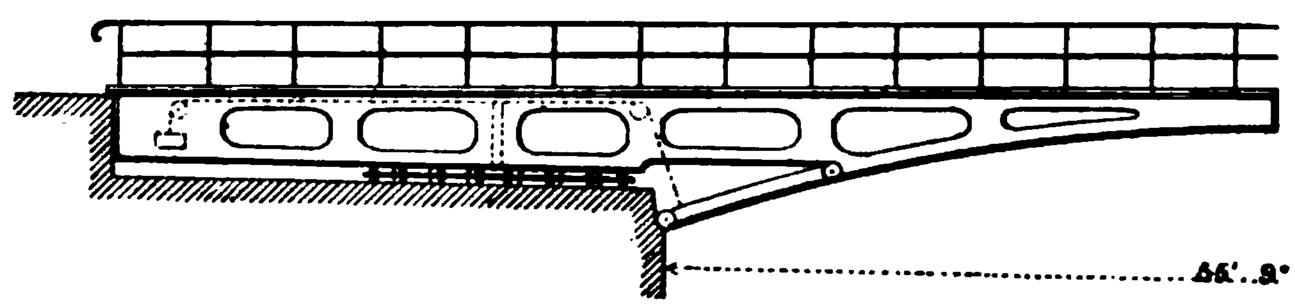
The bridges at Antwerp docks are generally swing bridges, one of which is exemplified in fig. 431, but there is a traversing bridge over the entrance lock to the Kattendyk Dock, which is constructed according to the type shown in figs. 432 and 433. The structure consists of two main plate girders of a uniform height of 9 feet, one on each side of the roadway, connected at intervals of 12 feet by cross joists, between which are rivetted

* Kinipple on "Greenock Harbour," *Min. Proc. Inst. C.E.*, vol. cxxx.; Macalister on "Caissons for Dock Entrances," *Min. Proc. Inst., C.E.*, vol. lxv.

† Vide "*Anvers, Port de Mer.*"

two series of longitudinal joists. A footwalk, 4 feet 6 inches wide, is carried on brackets outside the main girders. The width of the passage opening is 90 feet, and the total length of the bridge is 158 feet 6 inches. The roadway is paved with blocks of creosoted pine, laid upon a ½-inch bed of asphalt, which in its turn covers a floor of jointed oak. At the tail end of the bridge there is a counterbalance of 106 tons. The total weight of the movable platform is 370 tons.

To open the passage, it is necessary to lift the bridge to such a height as will enable the tail rollers to run back on the level of the roadway by which the bridge is approached, and this is effected by placing under each girder a hydraulic press with a large roller fixed on the head of the ram. The ram is 2 feet 7 inches in diameter and the roller 3 feet 7 inches; the latter is mounted on a 9-inch axle. The amount of lift is 3 feet. When the water enters the presses the bridge is lifted, but the tail end, which preponderates, does not begin to rise until the horn, or projection, at the nose end of the bridge comes in contact with a small inverted roller just below the surface of the coping. The tail end then ascends until the bridge becomes horizontal at its full elevation, as shown on the diagram by the dotted lines.



Scale: $\frac{1}{333}$.

Fig. 431.—Swing Bridge at Antwerp.

It is then drawn back upon the press rollers and the tail rollers by the action of a horizontal cylinder and ram, with chains and multiplying sheaves, situated beneath the bridge. The ram is 20 inches diameter with a stroke of 12 feet, and there are four sheaves at each end, multiplying the power eightfold. The chain is 1¼ inches diameter. An iron pathway, to bear upon each press roller, is fixed to the underside of each girder. The process of closing is the same as that just described, but in the inverse order.

The bridge was constructed by Messrs. Sir W. G. Armstrong & Co., of Newcastle, to the late senior partner of which firm the engineering profession is mainly indebted for the many present valuable applications of hydraulic power. Several bridges have been designed upon the same principle, and are reported to work very well, notwithstanding the excessive weight which is necessarily carried on the main rollers. In one case, where the arrangement is a little different from that described above, the load on each roller amounted to nearly 100 tons, and yet it was at work for more than 20 years without any important renewals of the working apparatus. In this instance both the rollers and the roller paths were of cast iron, 9 inches broad, the diameter of the roller being 3 feet.

Figs. 432 and 433.—Traversing Bridge at Antwerp.

Automatic cut-offs and other precautions are adopted to prevent any tendency to accident during the movement of the bridge.

The swing bridge shown in fig. 431 is notable for a lowering arm or strut designed to turn the bridge structure into an arch when in the closed position. The efficacy of the strut, however, as a compression member, is rendered dubious by the necessity for an accurate bearing, which cannot in all cases be ensured.

Bascule Bridges at Rotterdam.*

Between the years 1881 and 1894, a series of seven bascule bridges were constructed at Rotterdam, all upon the same principle, which is illustrated by the typical bridge (the Scheluwebrug) in fig. 434. Former bascules had been provided with a very considerable rise towards the centre of the span, in order to obtain as nearly as possible all the advantages of the arched system. This rise, however, proved very inconvenient for heavily-loaded vehicles, and in the later type the platform was made almost horizontal. In one instance only was this arrangement departed from, and in that instance the form of the bridge was parabolic at the haunches, with a very flat connecting curve in the centre of the span.

In the class of bridge under notice, the two leaves of which each bridge is composed, do not abut against one another at their junction. They are only connected in the closed condition by locking bolts, for the purpose of securing uniformity of pressure and deflection. The tail ends, however, derive considerable support from their abutment, when horizontal, against an iron structure placed above the watertight pits in which the tails revolve, and strongly anchored to the foundation. Each leaf, accordingly, is capable of acting as a self-sustained cantilever. The bridges are calculated to support a load of about 10 cwts. per square foot. The platforms are of oak, with a pavement of blocks of "djati," or teak.

Nearly all these bridges are moved by hydraulic power. The machines consist of oscillating cylinders receiving pressure from the town's ordinary water main by means of a hollow trunnion. The piston actuates a crank, which is in connection with the turning axis of the bridge.

An ingenious arrangement causes the withdrawal of the interlocking bolts to depend upon the closing of the two ends of the bridge to traffic by an iron grating, so that it is not possible to raise the bridge until this grating has been moved into position.

Appended is a list of the seven bridges with the principal particulars of their design attached:—

* Ysselsteyn on *Le Port de Rotterdam.*

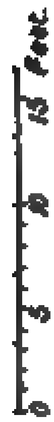


Fig. 434.—Bascule Bridge at Rotterdam.

	Name of Bridge.	Width of Platform.	Width of Passage.	Weight of Superstructure.	Remarks.
		Feet.	Feet.	Tons.	
1	Keizersbrug, .	23	33	135	Manual movement only.
2	Stokkenbrug, .	28	44	210	Movement by hand or hydraulic power.
3	Nieuwe Oostbrug,	25½	32½	146	Manual movement exclusively.
4	Jan Kuitenbrug, .	29	45	275	Hydraulic or hand power.
5	Spangaardsbrug, .	29	45	249	,, ,,
6	Nieuwe Leuvebrug,	32	47	322	Hydraulic power only.
7	Scheluwebrug, .	32	46	276	,, ,,

In addition to the foregoing, there is a bascule bridge across the entrance to the Binnenhaven, the span of which is 75 feet and the width of platform 34 feet. The upper surface is perfectly horizontal, but the four girders, of which each leaf is composed, are curved in form, and find a lower bearing 8 feet below the roadway level. The arched structure, however, has not been realised as designed. The union of the two extremities, in spite of several different devices successively tried, is not sufficiently perfect and each leaf remains a cantilever, exercising considerable force upon its axis, and causing a large annual expenditure for maintenance and repairs.

The bridge is twin, comprising two separate structures side by side, each capable of acting without the other in case of repairs, but under normal conditions coupled together.

The weight of each leaf is 121 tons, and gas engines supply the motive power.

Bascule Bridges at Chicago.*

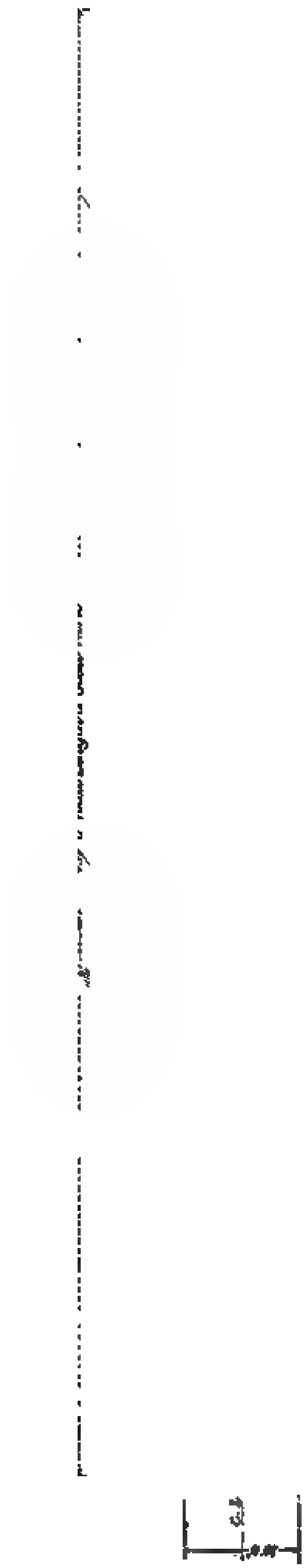
These are of the type described as rolling bascules—one of the latest examples of which, near Taylor Street, Chicago, is illustrated in figs. 435 and 436—a design due to the late Mr. William Scherzer. The heels or shore ends are fitted with curved and counterweighted girders, which roll on a path on the bridge abutment, the girders having holes fitting over the teeth of a horizontal rack, which serves to guide the motion of the bridge. Each bridge has two leaves.

The Van Buren Street bridge has a span of 115 feet between centres of bearings, and covers a waterway 109 feet wide. The structure is formed of three parallel trusses covered by a platform, comprising a roadway, 41 feet wide, and two footwalks, each 8 feet wide. The roadway accommodates a double track for electric trams.

The North Halsted Street bridge has a span of 127 feet and covers a waterway 121 feet wide. There are only two trusses in this case, the roadway being only 34 feet wide, with two footwalks, 7 feet 3 inches wide. Provision is made for an electric railway.

The railway bridge, between the two bridges just described, is constructed on the same lines. The span is 114 feet, and the channel width 108 feet. The bridge is composed of two similar or duplicate pairs of leaves

* Vide *Engineer*, November 26, 1897.



Figs. 435 and 436.—Rolling Bascules at Chicago.

placed side by side, each pair forming a complete span, and carrying a double railway track. Under normal conditions they are coupled together, but in case of repairs they can be disconnected, and each pair then acts independently of the other. Each leaf is so counterweighted that on drawing the centre and end locks, it rises to an angle of about 30°, rolling back on the abutment, and the application of power is only required to completely open the bridge or to close it. The weight of each double track is about 135 tons. The bridge can be opened or closed in thirty seconds, each leaf being operated by two horizontal struts connected to the ends of the trusses. The struts are run in and out by gearing, operated by a 25 H.P. electric motor. When the bridge is closed, each leaf acts as a cantilever, anchored by the tail end, which takes a bearing against the underside of the approach viaduct, the approach being firmly anchored to the masonry of the abutment. The end lock holds the tail firmly home against its bearing.

The following are the general dimensions of the bridge :—

Length between ends of approaches, ...	276 feet.		
Span between bearings, ...	114 ,,		
Width of channel, ...	108 ,,		
Headway at centre, ...	35 ,,		
Depth of truss at shore end, ...	26 ,,		
Depth of truss at free end, ...	6 ,,	6 inches.	
Width between trusses, ...	21 ,,	2	,,
Total width of bridge, ...	51 ,,	10	,,
Radius of heel of truss, ...	26 ,,		
Weight of each double track leaf, ...	135 tons.		
Total weight of bridge, ...	540 ,,		
Counterbalanced weight on each side, ...	28 ,,		

Swing Bridge at Marseilles.*

The bridge (figs. 437, 438, and 439) over the entrance to the Marseilles repairing docks has a total length of 203 feet 5 inches and a width of 46 feet. The framework consists of three parallel trellissed girders, each 11 feet 6 inches deep, with curved upper flanges. Between one pair of girders is a single line of railway; between the other pair a roadway, with a footpath, 6 feet 6 inches wide, carried on brackets outside the outer girder. The width of the waterway is 91 feet 10 inches, and the swinging bridge consists of two cantilevers, 126 feet and 77 feet 5 inches long respectively. The total weight of the structure is 700 tons, of which 125 tons is due to counterpoise. The bridge is raised and turned upon a hydraulic pivot of only 22·8 inches diameter, which necessitates a pressure of over 4,000 lbs. per square inch, obtained by means of a double-acting force pump and an accumulator. Each girder carries a roller under it near the extremity of its tail end; the three rollers are in a line parallel to the bearing girder, so that

* Gaudard on "Swing Bridges," *Min. Proc. Inst. C.E.*, vol. xlvii.; Price on "Movable Bridges," *Min. Proc. Inst. C.E.*, vol. lvii.; and Barret on "The Swing Bridge at Marseilles," *Min. Proc. Inst. C.E.*, vol. xlii.

Figs. 437, 438, and 439.—Swing Bridge at Marseilles.

the bridge may rest evenly on all three when slightly raised. This arrangement necessitates two circular roller paths of radii, 64 feet and 67 feet 7 inches, respectively. The bridge is turned by a chain passing round a cast-iron slewing drum, 46 feet in diameter, the motive power being supplied by two hydraulic cylinders, with rams, each 11·8 inches diameter, and 9 feet 2¼ inches stroke, one of which serves to open and the other to close the bridge.

The operation of turning consists in first releasing the wedges at the tail end, by which means the rollers at that part are lowered on to their tracks. The pivot press then lifts the bridge until the nose end is raised from its supports, and everything is ready for rotation. A hydraulic cylinder, 13·8 inches in diameter, actuates the wedging apparatus under a pressure of 700 lbs. per square inch, which is the same as that obtaining in the slewing cylinders. The kentledge is arranged to throw a weight of 15 tons on the guiding rollers while the bridge is being swung.

The pivot is enclosed in a press, 6·3 inches thick, which is secured by keys to a cast-iron base, from which it can be withdrawn for repairs. The prismatic top of the pivot inserts itself into a bearing plate fixed to the underside of the pivot girder. The surface of contact is made slightly convex, so that the bridge may always have a good bearing on its axis, despite any slight displacements during the process of lifting. A leather lining makes a watertight joint between piston and cylinder, but in order to prevent a tendency to tear from the turning stress imparted by the adherence of the rotating pivot, the interstice between the edges has been fitted with a band of india-rubber, which, by the interior adhesion it gives to the opposite edges of the leather, causes its exterior surface to slide on the metal. A horizontal sector is fastened to the head of the piston, which rests against two rollers with light movable axles, supported by a cast-iron bracket to counteract the lateral strain caused by the chains in turning the bridge.

Commenting in the *Annales des Ponts et Chaussées*, May, 1875, on the arrangements described above, M. Barret, then engineer to the Marseilles Dock Company, adds:—"If a similar bridge had to be constructed for an important line of railway, and over a channel through which there was a considerable traffic, it would be desirable to substitute a double line for the single tramway, and to make a footway on each side of the cart-road within the girders, which, though increasing the width to 59 feet, would make the bridge more symmetrical and easier to balance. The raising and lowering of the ends might be regulated by making the rollers at the tail end fall and rise in the roller boxes, keeping them always in contact with the roller paths by means of a counterpoise. The diameter of the piston (pivot) of the press might be increased to 4·9 feet, so that the bridge could be raised with the ordinary water pressure. The guide rollers might be increased in number, and placed higher up, so as to act all round the bearing plate. Also, if the webs of the girders were made of plate iron, the strains would be more evenly distributed, and the construction simplified with a slight

increase in the weight of the girders. With these modifications, it would be possible to construct swing bridges, weighing about 2,500 tons, which could be safely and easily worked."

Tilting Bridge at Marseilles.

At the Passage de La Joliette, 70 feet wide, at Marseilles,* there is a large traffic of unmasted timber lighters and but few sea-going ships. As it is therefore advisable to open the passage as seldom as possible for any considerable time, owing to the roadway traffic, a form of bridge (fig. 440) has been devised, combining the swinging principle with that of the bascule. For unmasted barges, the bridge is tilted by means of a piston

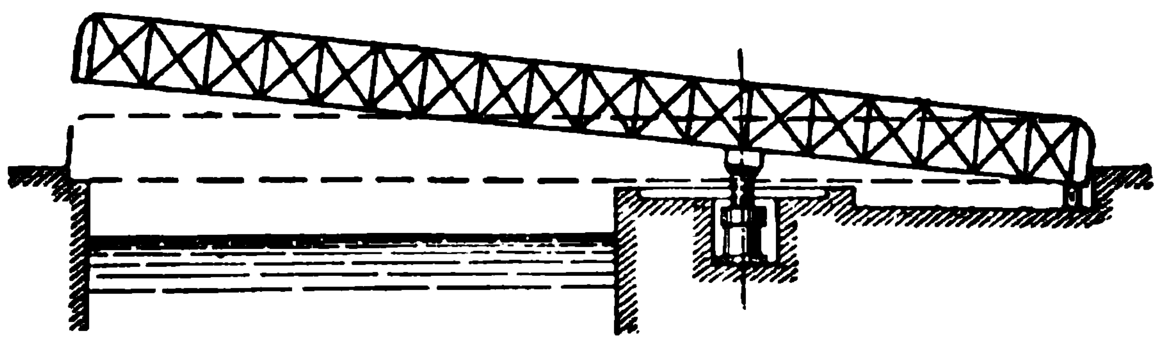


Fig. 440.—Tilting Bridge at Marseilles.

pivot, and it is only rotated for large vessels. When in the tilted position the gradient of the floor is 1 in 14 and a headway of 10 feet 3 inches is afforded. The time occupied in tilting is, of course, much less than in swinging.

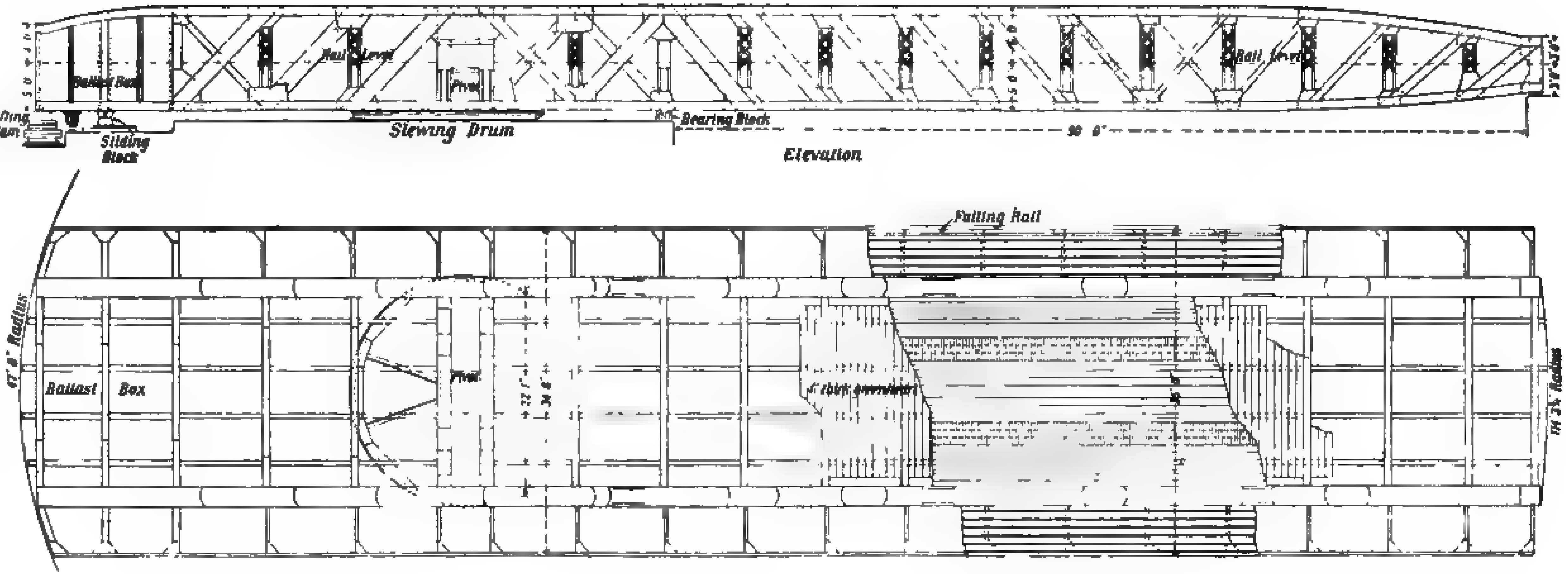
Single Swing Bridge at Liverpool.

This bridge (figs. 441, 442, and 443) constitutes a design used in three or four instances for spanning passages, 90 feet in width. The structure, which is of mild steel, consists of two main girders, each 159 feet long and 11 feet deep generally, but reduced to 6 feet in depth at the nose end. These girders are connected at intervals of 8 feet 6 inches by cross girders, 2 feet deep, supporting intermediate longitudinal joists, 12 inches by 6 inches. The pivot girder consists of two box girders, each 4 feet deep at the centre and 2 feet 9 inches deep at the ends, joined by $\frac{1}{2}$-inch diaphragms and a covering plate, the pivot casting being bolted to and between the box girders.

The main girders are 22 feet apart, centres, providing a double roadway, 17 feet wide, separated by a central cast-iron curb. A narrow space, for carters and others, adjoining each girder is also protected by a curb. The footwalks proper are two in number, each 6 feet 4 inches wide, and carried outside the main girders by brackets which are prolongations of the cross girders.

Although rails have not actually been laid down upon the bridge, provision has been made for their accommodation, by spacing the longitudinals to suit a double railway track, and the bridge has been calculated to sustain the heaviest type of locomotive as a continuous load.

* Price on "Movable Bridges," *Min. Proc. Inst. C.E.*, vol. lvii.



Figs. 441 and 442.—Swing Bridge at Liverpool.

Fig. 443.—Swing Bridge at Liverpool.

The decking is of greenheart, laid upon a 4-inch platform of creosoted pine. Under the wheel tracks, which are iron-plated, the greenheart is laid in longitudinal planks, 4 inches thick. The horse tracks, 3 feet wide, are of blocks, 9 inches by 5 inches by $3\frac{1}{2}$ inches, set in Portland cement. The footpaths are of 3-inch greenheart planks, laid longitudinally across $4\frac{1}{2}$-inch by $4\frac{1}{2}$-inch bearers. There are elm rubbers, $9\frac{1}{2}$ inches by $5\frac{1}{2}$ inches, at each side of the bridge. The handrail, which adjoins the waterway when the passage is open, is arranged to fall, so as to offer no obstruction to warps and lines.

While the passage is in use the bridge remains upon its pivot, but, having been rotated across to the closed position, a couple of vertical cast-iron rams, working in a 25-inch diameter hydraulic cylinder, with a 7 inches stroke, lift the extreme tail end of the bridge, so that the latter leaves the pivot and tilts forward on to bearing blocks at the edge of the coping on both sides of the passage. At the same time a pair of sliding blocks are brought under the tail end, and a very slight subsidence of the rams causes the bearing to be transferred to the blocks.

The slewing machinery consists of a pair of hydraulic rams, each 14 inches diameter, 9 feet 10 inches stroke, and furnished with sheaves giving a power of 2 to 1. The roller path is 43 feet radius and the wheels are of cast steel, 17 inches diameter, turned, bored, and coned. The slewing chain is $1\frac{5}{8}$ inches diameter. The radius of the slewing drum is 11 feet 9 inches.

The Victoria Swing Bridge at Leith.*

This bridge (fig. 444) constructed in 1874 has a clear span of 120 feet and was, at the time of its construction, the largest in the kingdom. The total length is 214 feet 3 inches, and the width over all, 39 feet 3 inches. The platform comprises two lines of railway and roadway, with a footpath on each side. The weight of the whole bridge is upwards of 600 tons, including a counterpoise of 240 tons. There are two main girders, each 27 feet in depth. The pivot or lifting press has a diameter of 5 feet 9 inches, and divides the bridge into a long arm of 147 feet and a short arm of 67 feet 3 inches.

The principle upon which the bridge is manœuvred is the same as that described in connection with the Marseilles bridge, with the exception that the ordinary hydraulic pressure of 750 lbs. per square inch serves to work the pivot without the intervention of a force pump. The turning gear is illustrated in fig. 445.

Swing Bridge at Stanley Dock, Liverpool.†

This bridge carries an overhead electric railway across the 50-foot entrance to the Stanley Dock. It is a combination of a swing bridge and a

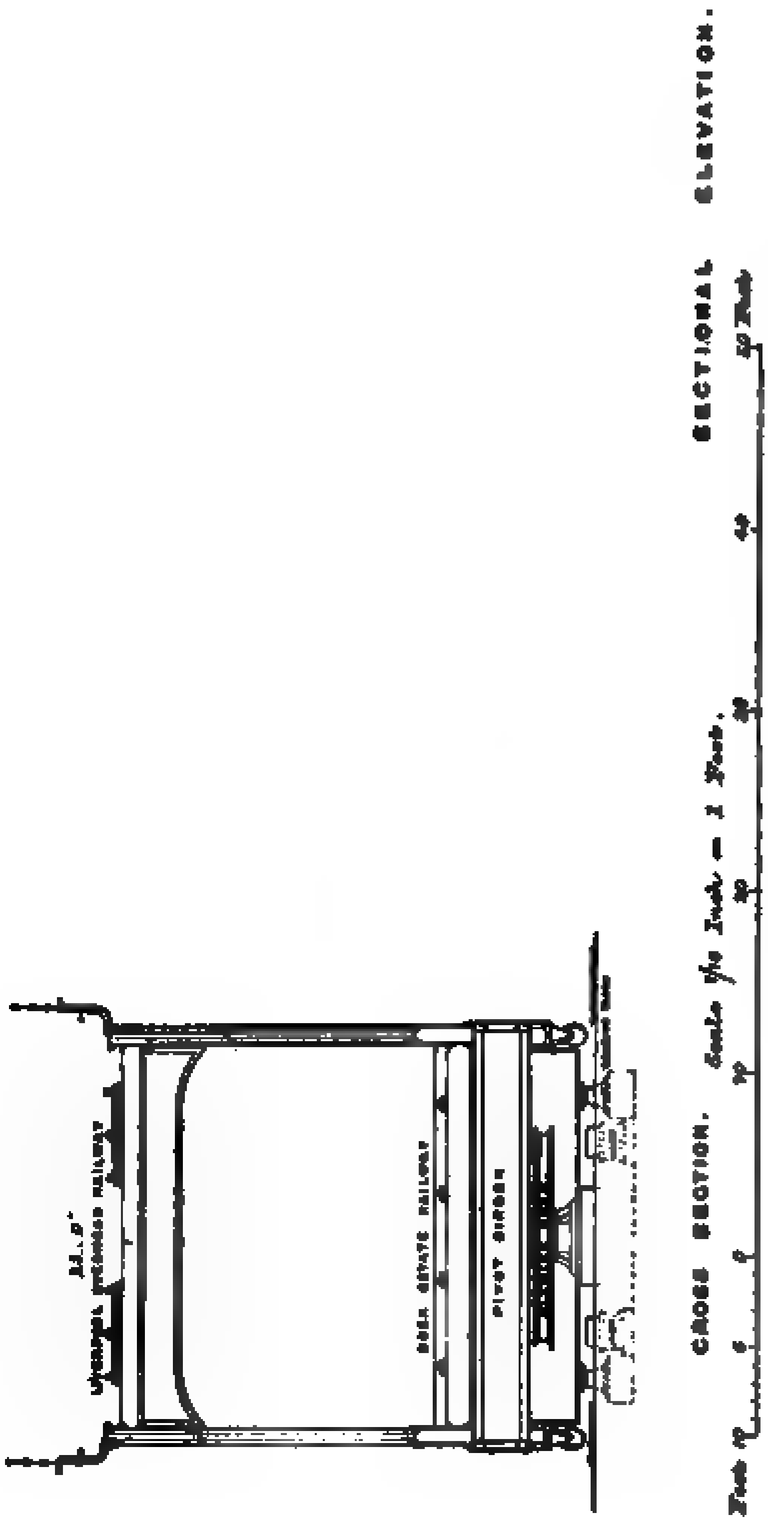
* Whyte, "Notes on Leith Docks and New Works in Progress," 1901.

† Greathead and Fox on "Liverpool Overhead Railway," *Min. Proc. Inst. C.E.*, vol. cxvii.

GENERAL PLAN

Fig. 446.—Swing Bridge at Liverpool.

drawbridge. It is in two leaves and has two decks. The upper carries the electric railway, the lower carries a double line of rails provided for the dock traffic. The lower level is arranged with bascule leaves, so that barges and small craft can use the passage without the necessity of swinging the whole structure and interrupting the railway service which is a very frequent one. The bridge is, in its normal condition, a fixed structure, resting upon bearing blocks at the tail end and upon two legs at the front of each abutment. To enable the bridge to be completely opened the following movements have to be made. The tail end of each leaf is slightly lifted to allow the bearing blocks to be withdrawn, and then it is lowered until it rests upon the roller path. In accomplishing this, the pivot of the bridge comes in contact with its socket, the girders are canted upward at the nose end, the intermediate supporting legs are lifted off their bearings, and the bridge is ready for swinging. The load on the pivot is 270 tons. The length of the bridge between pivot centres is 80 feet 6 inches, and 114 feet 6 inches is the extreme length. The width of way between the longitudinal girders is 21 feet. The slewing drum has a diameter of 12 feet, and is turned by a 1⅝-inch chain. The weight of the combined structure is 600 tons.

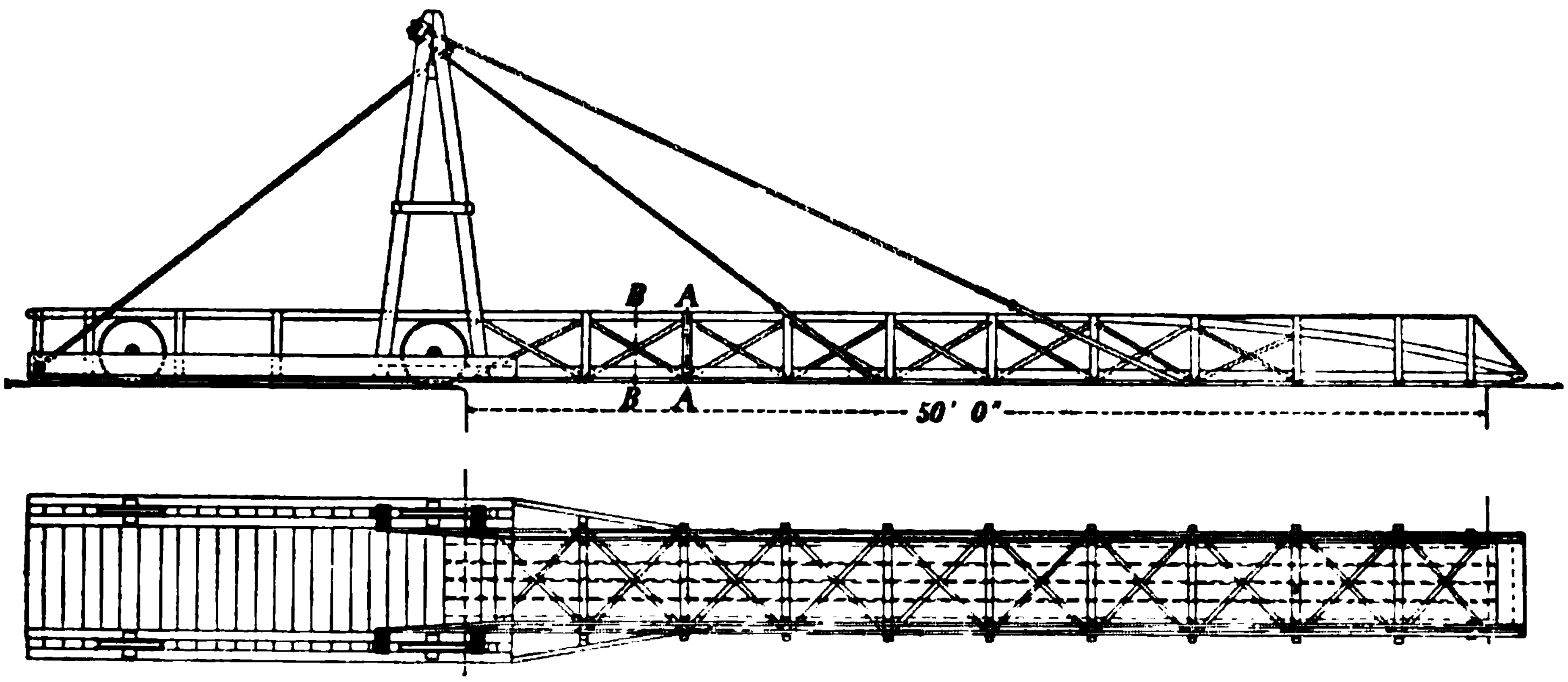


Figs. 447 and 448.—Swing Bridge at Liverpool.

Detailed drawings of the bridge are exhibited in figs. 446, 447, and 448.

Footbridges at Liverpool.

Illustrations are given of two types of footbridge—the first constructed in wood, and the second in iron. The wooden bridge (figs. 449, 450, and 451) which has a total length of 73 feet 6 inches, spans an opening of 50



Figs. 449 and 450.—Footbridge at Liverpool. Scale—16 feet = 1 inch.

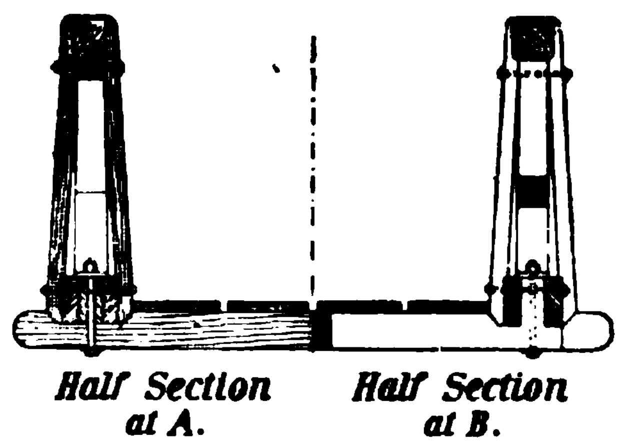
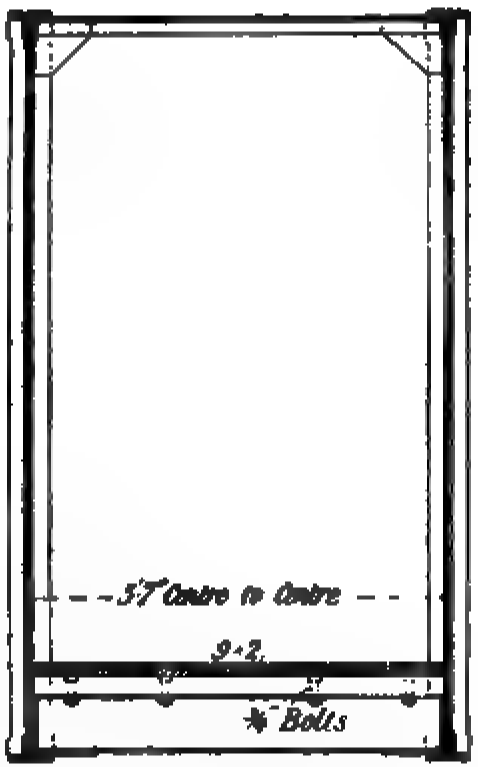
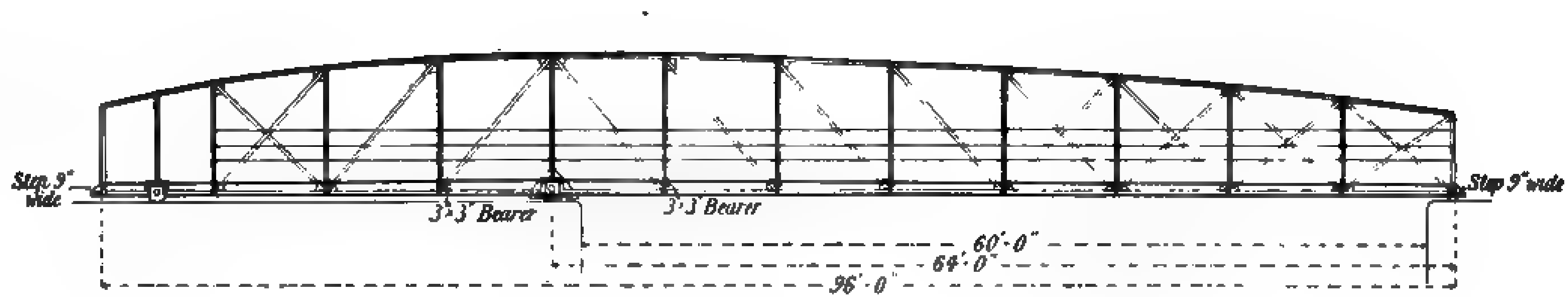


Fig. 451.—Footbridge at Liverpool. Scale—4 feet = 1 inch.

feet. The width of the footway is 4 feet, and in order to accommodate the ballast the bridge is widened at the tail end to 8 feet 2 inches over all. The ballast is composed of concrete laid in the floor and in the side panels. Movement is made entirely by hand. The iron bridge (figs. 452, 453, and 454) has a length of 96 feet, and covers an opening of 60 feet. It is propelled by hydraulic power, by means of rack and pinion gearing on the underside of the bridge floor. The bridge was tested as a cantilever with a uniformly-distributed load of 7½ tons.



CROSS SECTION.



ELEVATION.

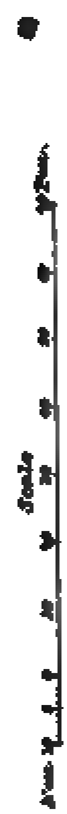
PLAN.

Figs. 452, 453, and 454.—Footbridge at Liverpool.

Note.—The top and bottom flanges consist of double angles, $3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$, and the verticals of double tees, $4'' \times 3'' \times \frac{3}{8}''$, generally. The inclined members are flat bars, from $3'' \times \frac{1}{2}''$ to $2\frac{1}{2}'' \times \frac{5}{16}''$. The deck bearers are $9'' \times 3''$ channels, with $3'' \times 3''$ wood grounds, and $9'' \times 2''$ planking.

Double Swing Bridge at Kidderpur.*

A double swing bridge, over passages 60 and 80 feet wide, has been constructed at the Kidderpur Docks, Calcutta, and is shown in figs. 455,



Figs. 455 and 456.—Double Swing Bridge at Calcutta.

456, and 457. The example is all the more interesting in that the axis of

* Bruce on "Kidderpur Docks, Calcutta," *Min. Proc. Inst. C.E.*, vol. cxxi.

the roadway, and therefore of the bridge, is not rectangular with the axis of the passages. This fact entails a greater length of bridge than would otherwise be necessary. The various details of construction will be readily understood from the diagrams.

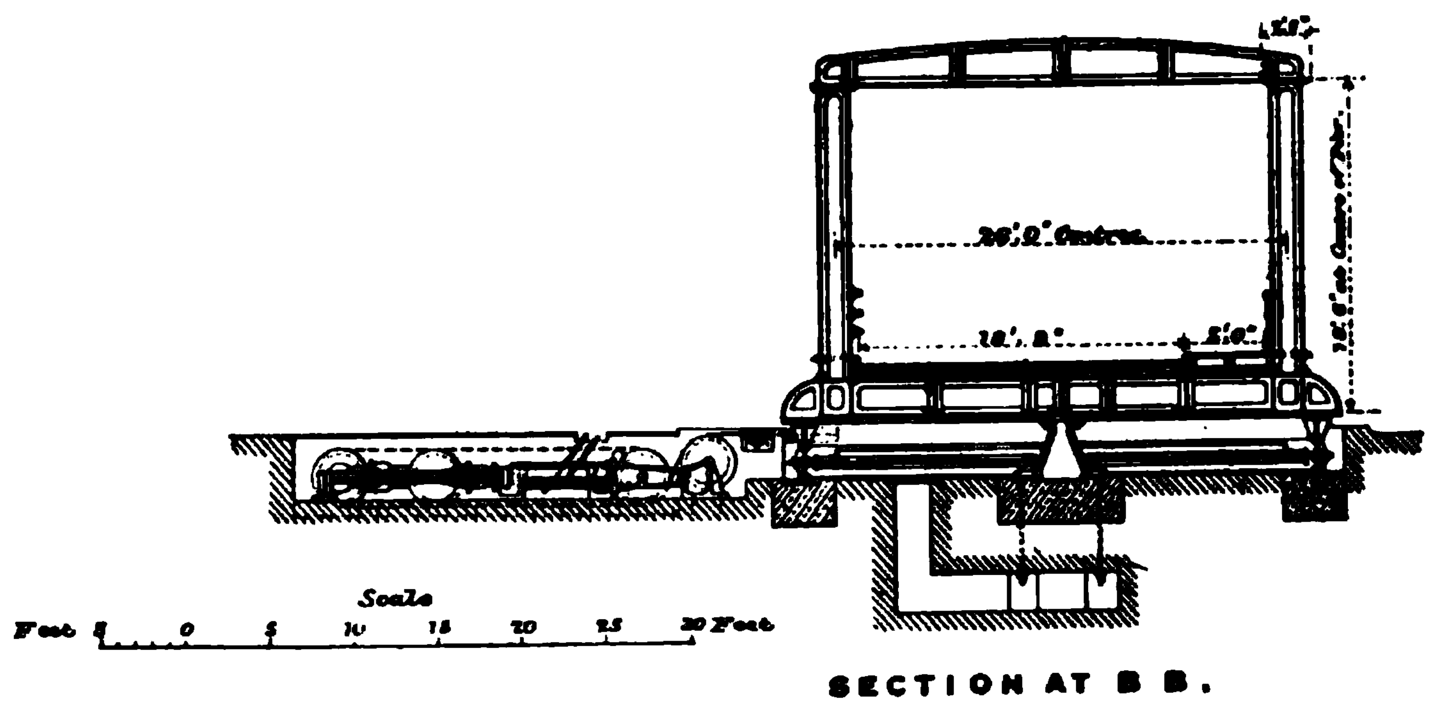


Fig. 457.—Swing Bridge at Calcutta.

Rolling Bridge at Greenock.*

A travelling bridge (fig. 458) on the same principle as a rolling caisson, connects the two sides of the entrance to the West Harbour at Greenock. The bridge has this difference, however, that it is constructed in openwork so as to allow the tide to pass freely in and out of the harbour.

The entrance is 103½ feet wide, and a bridge of this type was deemed most suitable for the site, owing to the great depth (60 feet below H.W.) to which it would have been necessary to go for a firm foundation for a swing bridge, apart from the inconvenience attaching to the accommodation of such a bridge upon a narrow quay. A timber gridiron, resting upon piles driven into the hard clay, and having their heads encased in plastic concrete, carries the rails (9 inches by 4 inches, solid section), which are laid to a 16-foot gauge upon greenheart runners at a depth of 26 feet below H.W. The bridge structure consists of three piers forming watertight tanks, each 18 feet by 18 feet, connected top and bottom by girders, 23 feet span. On the underside of the lower girders, six rollers are fixed at each pier. The bridge has a lowering deck similar to that already described (p. 360, *ante*). The total cost, including the hauling machinery, was £9,700.

* Kinipple on "Greenock Harbour," *Min. Proc. Inst. C.E.*, vol. cxxx.

Fig. 458.—Travelling Bridge at Greenock. Scale—16 feet = 1 inch.

CHAPTER XI.

GRAVING AND REPAIRING DOCKS.

Various Methods of effecting Repairs to Ships—Careening—Beaching—The Gridiron — The Slipway — The Hydraulic Lift — The Graving Dock — The Floating Dock—Essential Requirements of a Repairing Depôt—Comparison of the various Types in regard to Accessibility, Ventilation, Light, Capacity, Initial Cost, Maintenance and Repairs, Working Expenses, Durability and General Adaptability—Design and Construction of Slipways—Foundation—Permanent Way—Cradle—Sliding Slipways—Broadside Slipways—Stresses in Slipways—Design and Construction of Graving Docks—Types of Floating Docks—Process of Overhauling—Equipment of Repairing Docks—Distribution of Pressure on Keel Blocks—Description of Gridirons at Liverpool, Hydraulic Lift at London, Slipway at Dover, Graving Docks at Bremerhaven, Liverpool, Glasgow, Barry, and London, and Floating Docks at Cartagena and Bermuda.

The necessity of providing at every port sites, suitable in situation and equipment, where vessels can from time to time undergo examination, painting, and repair, is self-evident. There would be danger, to say nothing of loss of time and inconvenience, in transferring a disabled vessel from one port to another, however short the distance might be; and, apart from this, any lack of facilities for repair must inevitably react upon the prestige of a port and prejudice its development.

But, if the desirability of such a site be generally admitted, opinion upon the form it should take is not so unanimous. There are strong advocates for several very different types of repairing depôt. When we have examined the claims put forward in favour of each of these, we may possibly be able to assign some order to their respective merits.

Apart from the operation of **careening**, in which a water-borne vessel was temporarily given a pronounced list, the earliest means of obtaining access to the under side of a ship was that of dragging it by hand out of the water on to some moderately sloping strand of firm sand or gravel. If too heavy for manual haulage, the vessel was caused to take ground at high water, so that the receding tide left her high and dry. Such was the method of **beaching** as practised by the Phœnicians, the Egyptians, and other nations during the infancy of the mercantile marine. For light vessels of shallow draught the method is, no doubt, quite satisfactory and sufficient, and, despite its primitive nature, it is still in use at the commencement of the 20th century. Its modern prototype is the **Gridiron**, located in a tidal basin, and consisting of an extended series of parallel beams or logs laid at regular intervals upon a firm masonry foundation. The operation is simply

to float the vessel into position and leave her suitably moored; the tide does the rest.

This system, though simple and effective in its way, has many defects. In the first place, it is only practicable in localities where there is sufficient range of tide for the purpose. Then repairing operations are intermittent and have to be suspended with each recurring period of high water, occasioning delay and the repetition of manœuvres. And lastly, the floor, in the absence of any means of adjustment to the keel of the superimposed vessel, does not lend itself to any but the rudest kind of support.

The Carthaginians seem to have discovered an improved method of dealing with the problem by the introduction of **Artificial Slipways**, in which smooth timber slides formed a less frictional surface for the haulage of ships than the rough and irregular contour of a natural beach. Furthermore, they had the decided advantage of being utilisable in almost any situation. This was the origin of the modern slipway and slip-dock. The design has naturally undergone many modifications and improvements since the days of triremes and galleys, and it now exists in several distinct forms, but it is still essentially the same design. It would, of course, be superfluous to trace the various stages of its development, and we need only concern ourselves with the features displayed by its representative of the present day. Long timber ways, carrying iron rails, are laid at a uniform slope, ranging in different cases from about 1 in 15 to about 1 in 25, from some distance under water to a point at which the longest vessel to be accommodated is completely out of the range of the tide. A cradle or travelling frame is passed down the ways and under the oncoming vessel's keel. The latter takes a bearing upon the cradle, which is then drawn up to the highest point by suitable hauling gear. Despite its advantages, the drawbacks to the system are sufficient to prevent its general adoption. The length of a slipway is necessarily great, on account of its prolongation under water to a depth equal to the draught of vessels using it. This entails the appropriation of valuable water space and offers obstruction to navigation. To obviate these ill effects to some degree, the cradle has been made telescopic or collapsible, so that it consists of sections attached to one another by sliding bars. These sections, compressed at the foot of the slipway, are drawn out to their full extent by the hauling apparatus as each portion receives its proportionate load. The percentage of length, however, saved by this device is small. The appropriation of land space in congested districts is also an expensive matter, and recognition of this fact has led to the introduction of side walls and a pair of watertight gates at low-water level. The ship has then only to be withdrawn within the gates which shut out the tide. In this respect the slipway trenches upon the province of the graving dock and becomes a **Slip-dock**.

To do away with the excessive length of a slipway, the **Hydraulic Lift** was devised, towards the middle of last century, by the late Edwin Clark. In some respects it is akin to the gridiron, consisting of a horizontal platform

upon which a vessel can be floated. Here, however, the resemblance ceases, for in this case the platform is formed of pontoons, the whole of which are raised by hydraulic pressure until the vessel is entirely above water. The operation, in fact, produces the effect of a falling tide and avoids the inconvenience of a rising one. The whole structure remains afloat until the time comes for the vessel to be re-launched.

We now come to the **Dry** or **Graving Dock**, the principle of which is the reverse of those already described; instead of withdrawing the ship from the water, the water is withdrawn from the ship. In its earlier stages, it is but the natural and logical development of the beaching process. Finding the inconveniences of only having access to their vessels during short periods at low water, the obvious advantage of enclosing them within temporary mounds or banks of earth would suggest itself to enterprising shipwrights of ancient times. Then, in order to reduce the labour of constructing a continuous dam, the selection of a natural creek or inlet would occur, involving a dam across only one end. From a natural creek to an artificial chamber is but a single step, though, no doubt, some time would intervene between the two stages.

A modern graving dock is an excavated chamber, three sides and the floor of which are lined, either naturally or artificially, with watertight material. The fourth side, or end, is the entrance, and is provided with a pair of gates or a caisson. After the entry of a ship, the entrance is closed and the water is pumped from within the dock, though in certain cases the operation may be partially effected or, at any rate, assisted by the fall of the tide.

Lastly, we have the **Floating Dock**—a hollow structure, formerly of wood but now universally of iron or steel, generally similar to a graving dock in outline, but gradually diverging therefrom in process of evolution, and entirely dissimilar in action, in that it reverts to the former principle of withdrawing the vessel from the water. It is, in fact, an outcome of the hydraulic lift. To receive its charge the floating dock is sunk to the requisite depth by allowing its air chambers to fill with water, which is afterwards removed by pumping when the vessel has been berthed. This process causes the dock to rise bodily and, in so doing, to lift the vessel above the water line.

Thus far we have very briefly reviewed the rise and progress of various repairing systems. We will now proceed to consider them more closely with reference to their construction and equipment. But, before doing so, it will be well to lay down three general essential requirements of any system :—

1. **Accessibility.**—All parts of a vessel's keel and under side must be readily accessible. Beaching is deficient in this respect, unless the position of the vessel be changed, and this is not always feasible.

2. **Ventilation.**—If a vessel has to be painted, it is essential that her sides should dry as quickly as possible, and this result is best achieved in

the open. Hence gridirons, slipways, and lifts have a certain advantage over docks, and, of the latter, the floating dock is more open than the graving dock.

3. **Light.**—Artificial light can, of course, be provided, but natural light is always better and more economical. The same comparison holds good as in the case of ventilation.

Apart from these general requirements, there are various points of view from which the advantages of the systems may be estimated, and, accordingly, we will deal with these in order. Setting aside the gridiron as too primitive and the hydraulic lift as now superseded by its development, the floating dock, we may usefully confine our comparison to the remaining three types.

4. **Capacity.**—Although no apparent limitation attaches to the size of slipways, yet it will be found that they have only been constructed for a comparatively small class of vessel—those with lengths not exceeding 350 feet and dead weights of not more than 5,000 tons. This arises from three causes: first, the excessive length of slipway, both above and below water, required for the reception of larger ships; secondly, the liability of such ships to undergo strain during the process of getting them on to the cradle; and, thirdly, the difficulty of keeping a very large slipway remuneratively engaged. Theoretically, there is no reason why a ship of any length and weight should not be supported upon a slipway of sufficient size and stability, and to economical reasons alone must be attributed the main objection to its more extended utility.

Judging from existing examples, the size of graving and floating docks is restricted by no such consideration, and their maximum capacity has yet to be determined. Every succeeding year witnesses an increase in dimensions. As regards their relative capacities there is some difficulty in instituting a comparison, for that of a graving dock is based upon its linear dimensions, the weight of any incoming vessel not entering into account, while a floating dock, open at each end, is gauged by the weight which it can lift, and is practically independent of size. The largest vessels, designed or in existence at the present time, are nearing or have reached a length of 760 feet, a beam of 78 feet, a loaded draught of 36 feet, and a displacement of upwards of 38,000 tons. The largest graving docks have lengths of over 850 feet, entrances more than 85 feet wide, and a draught of water on sill at high water of ordinary spring tides somewhat exceeding 32 feet. While the superficial area of such graving docks is largely in excess of all present requirements, it will be noticed that there is an apparent insufficiency in draught, and this fact is often alleged as a disqualification. But in the great majority of cases, a vessel will discharge the whole or the larger part of her cargo before entering the dock and so reduce her draught by several feet. At the same time, it must be admitted that the margin thus obtained is by no means a large one, and it frequently disappears at neap tides, while there is always the remote contingency of a seriously damaged vessel having to be docked fully loaded immediately upon its arrival at a port. It is an

unfortunate feature of graving dock construction that an extra foot in depth adds most disproportionately to the cost.

The largest floating dock at present in existence has a lifting power of 18,000 tons, or about one-half of maximum requirements. The increase in size of late years has, however, been so rapid that there is every probability of the disparity being cancelled in a very short time. It has, moreover, been justifiably pointed out that, whereas a graving dock is unable to accommodate a vessel any one of whose dimensions exceeds a certain limit, a floating dock, on the other hand, is quite capable of partially raising a heavier vessel than she has been designed to lift entirely above water. A floating dock at Barrow, with a lifting power of little more than 3,000 tons and a length of 242 feet, raised the s.s. "Empress of China," 485 feet long and 4,500 tons displacement, sufficiently high to allow her propellers to be removed and replaced. Any excessive overhang, however, is liable to cause severe strain both in the ship and the dock, and it is inadvisable to risk carrying such an experiment too far.

The following table affords some particulars of the largest existing ships:—

TABLE XXXV.—Particulars of Some of the Largest Modern Vessels.

Vessel.	Line.	Date of Construction.	Extreme Length.	Breadth.	Moulded Depth.	Draught.	Gross Tonnage.	Displacement.
			Feet.	Feet.	Feet.	Feet.	Tons.	Tons.
Baltic, ...	White Star, . .	1903	725·7	75	49	....	23,000	40,000
Cedric, ...	,, . .	1902	700	75	49·3	36·5	21,000	38,200
Kaiser Wilhelm II., .	North German Lloyd,	1902	706·6	72	52·6	29	20,000	26,000
Kronprinz Wilhelm, .	,,	1901	663	66	43	29	15,000	21,300
Celtic, ...	White Star, . .	1901	700	75	49	36	20,880	37,700
Deutschland, . . .	Hamburg American,	1900	684	67	44	29	16,502	23,620
La Lorraine, La Savoie, .	French Transatlantic,	1900	582·4	60·6	39·4	25·6	11,869	15,400
Oceanic, ...	White Star, . .	1899	704	68·3	49	32·5	17,274	28,500
Kaiser Wilhelm der Grosse,	North German Lloyd,	1898	648·7	66	43	28	14,349	20,880
St. Paul, St. Louis, .	American, . .	1895	554·2	63	42	26	11,629	16,000
Campania, Lucania, .	Cunard, . . .	1893	622	65·3	41·6	25	12,500	18,000

5. **Initial Cost.**—The factor of locality enters so largely into the question of cost of construction of slipways, that it is impossible to fix any absolute standard of comparison. For example, a slipway at Penarth, built in 1879, and capable of accommodating a vessel of 2,500 tons deadweight, cost £25,000 or £10 per ton. On the same basis, a slipway at Belfast, constructed in 1847, for vessels of 1,000 tons, should only have cost £10,000, whereas it cost £17,000 or nearly double that amount, of which £12,000 was spent on foundations alone. Mr. Walter Beer * estimates the cost of slipways for small boats of 600 tons at £9,000 or £15 per ton—an intermediate value to the previous cases. Not much reliance, therefore,

* Beer on "Ship Slipways," *Min. Proc. Inst. C.E.*, vol. cxviii.

could be placed upon an estimate for a slipway to accommodate modern ships of from 20,000 to 30,000 tons, with lengths of 600 to 700 feet, especially when the largest slipway in existence is one with a cradle of only 330 feet and a power of 5,000 tons.

So, too, with graving docks. One at Newport (Mon.), built in 1890, cost £70 per lineal foot, or 10s. per square yard of internal cross-section below high water; another at Biloela, New South Wales, completed about the same date, cost £440 10s. per lineal foot, or 26s. per square yard of section; for a third at Halifax, finished in 1889, the figures were £233 and 15s. 6d. respectively. No useful purpose, accordingly, can be served by attempting to fix the unit of expenditure. The kind of material (whether concrete, timber, brickwork, or masonry), the mode of construction, the nature of the foundation, the state of the labour market, and the cost of transport—all these conflicting conditions combine to render nugatory calculations based on existing data.

The fluctuation in the price of iron and steel, more than anything else, influences the cost of a floating dock, but there are often special features to be taken into account. Not infrequently a site has to be specially prepared by dredging for its reception. Shore connections and approaches are required, more particularly for the type known as the "off-shore dock." Also for docks built in this country, to be located at Colonial or Continental ports, there is the cost of freight or of towage and insurance.

Herr Howaldt* of Kiel, estimates the cost of composite floating docks of wood and iron, designed on his system, at 110s. to 120s. per ton of lifting power, if built in the west of Europe, and at 170s. to 200s. per ton if built in the east of Europe.

For docks altogether of iron, he estimates the cost at 180s. to 200s. and 230s. to 270s. per ton of lifting power, in the west and east of Europe respectively. Messrs. Clark and Standfield state "an all-round figure of £10 per ton of lifting power for floating docks of medium size."

At first sight it may appear that the cost of a light, hollow iron structure, built amid the conveniences of the shipbuilding yard, must inevitably be less than that of a masonry or concrete dock, involving a deep excavation, with expensive gates and other appurtenances. Such, however, is not necessarily the case. Undoubtedly, there are circumstances of site and foundation which would render the construction of a graving dock an inadvisable, if not an impossible, proceeding, but it is not improbable that the same conditions would equally preclude the construction of such essential adjuncts to a floating dock as a jetty and a shipbuilding yard. These circumstances are generally abnormal and, in the main, local conditions are favourable to either type.

Speaking roughly, but upon a basis of experience, the cost of a graving dock constructed in this country, under normal conditions, to accommodate a vessel 700 feet in length, should not greatly exceed £200,000. The

* Howaldt on "Floating Docks," *Int. Nav. Cong.*, Dusseldorf, 1902.

displacement of a vessel of this size would be at least 25,000 tons, and, according to Herr Howaldt and Messrs. Clark and Standfield alike, the cost of a floating dock to receive her would lie between £225,000 and £250,000. Or, looking at the matter another way, the proportion of deadweight of a floating dock to lifting power, from a number of examples, averaging about 45 to 100, the deadweight of a floating dock as above would be 11,250 tons, which at, say, £20 per ton (to include all fittings and pumping machinery), comes to £225,000 as before. This is without taking into consideration any ancillary works, such as shore connections, site dredging, &c. So that, as regards the cost of the largest docks, the balance inclines in favour of the graving dock. This opinion receives confirmation in the report of the engineer (Mr. Wm. Ferguson) to the Port of Wellington, N.Z., who after a tour of inspection of the repairing docks in Great Britain, Australia, and the United States, recommends the adoption of a concrete graving dock for that port as less expensive than a floating dock of similar capacity.*

6. **Maintenance and Repairs.**—The structures of slipways and graving docks, if solidly built in the first instance, require very little attention afterwards, whereas owing to the destructive action of salt water on ironwork, floating docks call for regular inspection and frequent painting. In slipways the cradle wheels occasionally get broken, but this item should equitably be included in repairs to machinery, which are common to all three types, though possibly a more pronounced item in floating docks. The structural repairs of a concrete or masonry graving dock, with greenheart gates, are infinitesimal. If iron gates are used, they will necessitate some expense of upkeep, as against a reduction in their cost of construction compared with wooden gates. No doubt, the timber graving docks prevalent in the United States require extensive repairs from time to time, but in this case also, the capitalised amount is balanced by a corresponding economy in initial expenditure, and they represent, moreover, a very limited class.

According to some statistics, supplied by Messrs. Clark and Standfield, the average annual cost of upkeep of iron floating docks ranges between ·75 and 1·5 per cent. of the invested capital. The former figure represents exceptional care in primary preparation, the outside surfaces being particularly well painted and "the whole of the mill-scale having sweated off before launching, so that the paint was fairly on the iron."

7. **Working Expenses.**—In this respect the floating dock exhibits an economy far beyond that of the graving dock, because, in the former case, the quantity of water to be removed by pumping is little more than the actual displacement of the vessel which is being docked, while in the latter case, unless any assistance can be rendered by a falling tide, the volume of water to be pumped out is the cubic contents of the graving dock, less the displacement of the ship. Furthermore, while for a graving dock the

* *Report on Docking Facilities for the Port of Wellington*, 1901.

amount of pumping increases with a decrease in the size of the vessel, for a floating dock the reverse is the case, since it need only be sunk to a depth sufficient to take the vessel's keel. There is only one point which slightly reduces the overwhelming advantage of the floating dock, and that is in reference to the head pumped against. In the case of the graving dock, the head varies from zero to the depth of the floor below free water level, and the mean head may be approximately stated at one-half this depth. In the case of the floating dock, the initial head is the draught of the vessel plus the depth of the floor pontoon, and the final head is the latter of these two amounts. Hence, supposing two vessels of equal draught taken, the one on to a floating dock and the other into a graving dock, the depth of water in the docks being likewise the same, then the mean head of pumping in the former instance would exceed that in the latter by one-half the depth of the floor pontoons. But this advantage is more apparent than real, for it only occurs in the isolated case of a vessel of maximum draught using the graving dock. In the majority of cases the clearance between keel and floor is much greater than the semi-depth of a floating pontoon.

From a specific comparison between two docks of equal capacity, it has been found that the pumping power required for the graving dock was nearly four times that required for the floating dock, the duration of pumping being the same in both cases. If the power had been equalised by differentiating the time, the excess consumption of fuel and oil would still have been retained. Again, apart from the primary emptying of a graving dock, an auxiliary drainage pump is required to deal with leakages. In a floating dock there is no leakage, and, therefore, no necessity for a drainage pump.

On the other hand, it must not be overlooked that, the main pumps being only intermittently employed, it is quite feasible for a single pumping station to serve two or more graving docks, whereas each floating dock requires its own pumping plant, and this is often subdivided and distributed throughout the dock. Again, on account of the necessity of maintaining equilibrium in the floating dock, great care has to be exercised and attention paid to numerous valves. This entails a large working staff.

8. **Durability.**—Here the balance of merit reverts to the granite, brickwork, or concrete graving dock, which is practically indestructible.

The life of an iron or steel floating dock depends naturally on the care which is devoted to its maintenance, and upon the locality in which it is placed. In Chap. viii., it has been stated that a pair of iron gates, under average conditions, may be expected to last thirty years, but as overhauling and repairing can be carried out much more effectively, and with greater facility in the case of a self-docking floating dock, these more favourable conditions warrant the expectation of somewhat greater longevity—say forty or forty-five years.

The Bermuda Dock, launched in 1868, was found to have suffered

considerably by the end of the century, and other docks of the non-self-docking type have undergone equally rapid deterioration; but, on the other hand, the Cartagena Dock, built in 1859, is still in good repair, as also are the pontoons of the Victoria Dock, constructed in 1857.

A timber graving dock must necessarily be very liable to decay owing to its alternate exposure to the wet and the dry condition.

It has been pertinently pointed out that a dock may outlast its period of usefulness; that, with the rapid increase in size and alteration in shape of modern ships, a repairing dock ultimately becomes incapable of receiving any but those which are obsolete. This may be true to some extent, but it is no less true that both graving and floating docks are capable of being altered within certain limits, so as to adapt themselves to new conditions. They have been lengthened in more than one instance. Any increment in width and depth, however, can only be obtained at practically prohibitive expense, and the author is only aware of a very few instances in which such alterations have been carried out. The cutting away, in some cases, of the lowermost altar-courses of masonry docks has produced an additional few feet of bottom-width at a moderate cost.

9. **General Adaptability.**—There are several detached points of practical importance which may be grouped under the above heading.

(1) A floating dock has the advantage of mobility. It may be towed to another port. *Per contra* it may founder or suffer shipwreck.

(2) A floating dock may conceivably be trimmed by water ballasting to take a ship with a list so pronounced that it could not pass through the vertical profile of a graving dock entrance. Practically, such a step would be attended with serious risk of capsizing.

(3) Accidents are more rare in graving docks. Floating docks have sunk under ships of heavy tonnage, though not, it must be admitted, in recent times or with docks of the latest type.

(4) A floating dock takes comparatively little time to construct—say, from seven to nine months with expedition. An average graving dock could hardly, under the most favourable circumstances, be built in less than two years.

(5) Where land is dear, or the site restricted, a floating dock either renders its purchase needless or allows of its allocation to other purposes.

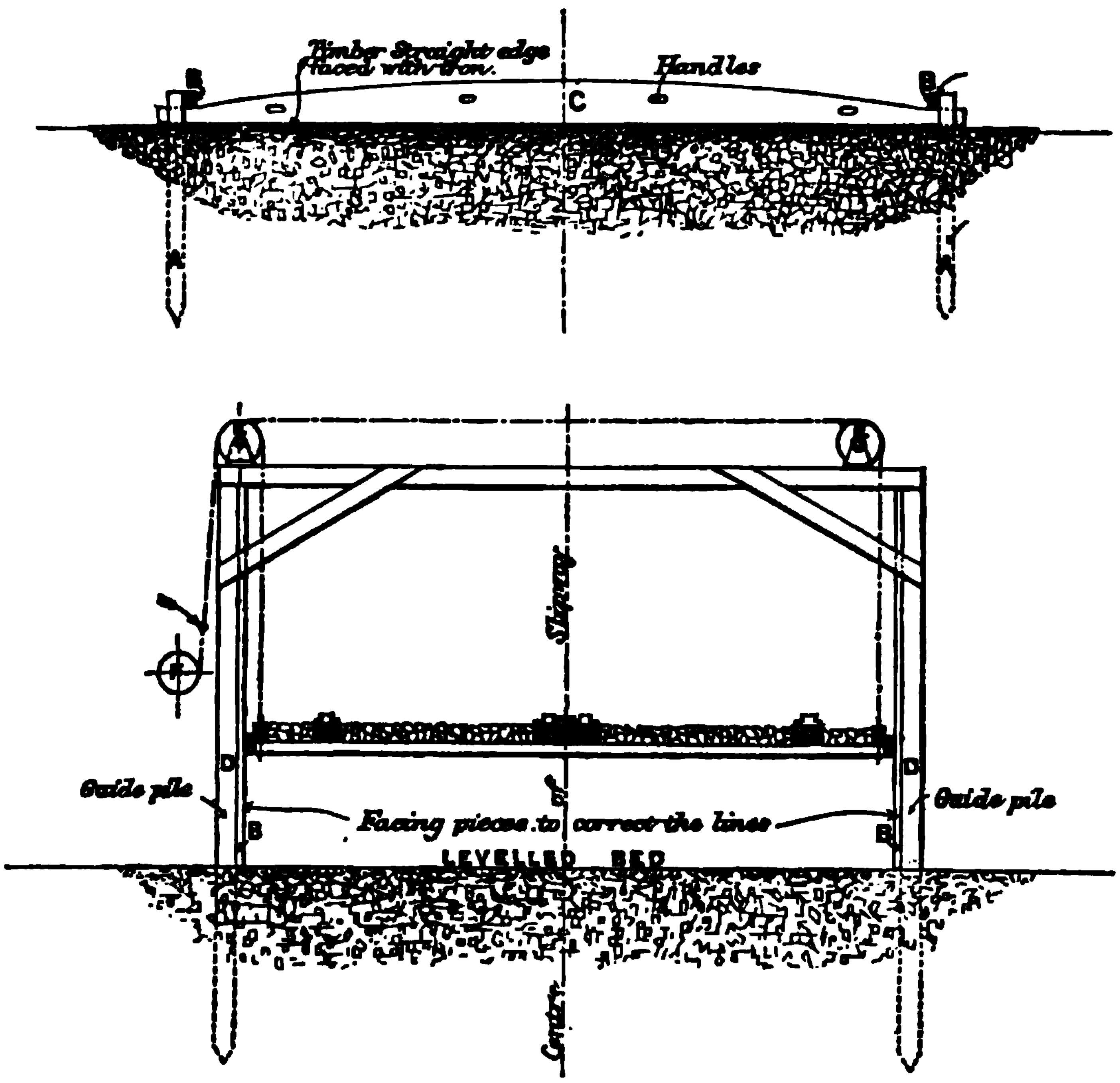
Design and Construction of Slipways.

The essential parts of a slipway are:—(1) The foundation, (2) the permanent way, (3) the cradle, and (4) the hauling machinery.

The **Foundation** should, if possible, be absolutely incompressible; but, failing that ideal, a very slight settlement is permissible, provided it be uniform throughout. Any transition from an elastic to a rigid base, or *vice versâ*, throws considerable local strain upon the cradle, often resulting in broken rollers. The intensity of pressure on slipways is not great, the

superimposed weight being spread over a large area. In the largest slipways at present in existence, the weight of vessel and cradle does not exceed 10 tons per foot run, and in smaller slipways, it may be taken at one-fourth less. Accordingly, where the ground is naturally very firm, little more than mere surface dressing will be requisite, with perhaps a shallow bed of concrete. In other instances, the site may require some dredging and subsequent levelling with rubble filling to the under side of a concrete bed, but in all cases of uncertain strata, bearing piles should be resorted to.

A very considerable portion of a slipway is necessarily under water, and the construction of this section often presents some difficulty. At places where there is a great range of tide, opportunities are afforded at low water for getting the bulk of the work done without serious inconvenience. On



Figs. 459 and 460.—Slipway Construction.

the other hand, where the tidal range is small, a temporary dam for the exclusion of water from the site becomes a desideratum, if not a necessity. The expense attending this mode of procedure is a deterrent to its ready adoption. Under favourable circumstances, the work may be economically carried out by divers in a sufficiently satisfactory manner. The following is an account of the system as practised by Mr. John Thompson :—*

"When the portion of the site below low water had been dredged out to the desired depth, the foundation was made by filling in broken stone of

* Lightfoot and Thompson on "Slipways for Ships," *Min. Proc. Inst. C.E.*, vol. lxxii.

convenient size to near the level of the intended platform. Upon this a layer of macadam was placed, bringing the foundation up to the required height. As a guide for the accurate execution of this work, a line of piles, A (fig. 459), was driven on each side of the foundation, clear of the sides of the timber platform, and to these piles, guide timbers, B, were affixed at the required inclination of the slipway and at the depth of the ends of the straight-edge above it. The foundation was now ready to be dressed off true by divers, who, as they frequently had to work in the dark, were provided with iron-faced straight-edges, C, made about the weight of a similar volume of water so as to be easily moved. These were long enough to reach across the entire foundation and to slide underneath the guide timbers. With these straight-edges, the divers were able to dress the macadam face so truly that, in one case of a foundation, 360 feet long, it was found, after the platform was finished, there was only one error of $\frac{3}{16}$ inch."

In the construction of a slipway for Earle's Shipbuilding Co., at Hull, by Mr. Godfrey, in 1882, the method of piling the foundation was adopted. "Whole timber piles, cross sleepers, and longitudinal bearers were used throughout. In the centre way, two rows of piles were driven, 18 inches apart from centre to centre, transversely and 3 feet from centre to centre, longitudinally. For the side ways single piles were driven, 6 feet from centre to centre, these coming opposite every second row of piles in the centre way, thus giving one pile for each lineal foot, or a supporting power of 10 or 12 tons per lineal foot. A sleeper, 30 feet long, was placed transversely on the four piles and one, 6 feet long, on the two intermediate piles. Upon these sleepers were fixed the longitudinal timbers or rail bearers, securely fastened with oak trenails. The centre timbers were 4 feet 6 inches wide to take a plate of the same dimensions. The ground for 4 feet below the cross sleepers was excavated and filled with rough chalk for a width of 15 feet on both sides of the slipway; the whole was planked over with 3-inch redwood deals." The piling was effected as follows:—A cofferdam could not be thought of, the situation being too exposed and the method too costly. The width of the slipway being 30 feet, a traveller 35 feet wide was constructed to span it transversely and placed upon a line of rails, the diameter of the wheels being made to suit the inclination of the slipway. Upon this traveller was placed a Sissons & White's steam pile-driver, with 40 feet leaders, and a ram weighing 21 cwts. As the tide ebbed the pile-driver was allowed to go down upon the traveller by gravitation, and the piles were driven in successive rows of two and four alternately, the machine being worked across the traveller from side to side. When the tide rose, the traveller was withdrawn to the higher portion of the work.

The Permanent Way is usually laid at some gradient between 1 in 15 and 1 in 25. There is a slipway at Palermo with a gradient of 1 in 13·3, but this is exceptionally steep, the average being 1 in 20. Any flatter slope than 1 in 25 causes an unnecessarily great length of slipway. Occasionally curved slipways may be found with a steep inclination below the water-line,

gradually becoming flatter as the summit is reached. The permanent way generally consists of three or four main lines of rails, arranged in pairs close together, the rails being of a shallow type, with 3 to 6 inches flat bearing surface. Between the centre pair is a strong cast-iron rack to receive the pawls of the cradle. The rails are spiked to longitudinal sleepers which, in their turn, are carried by cross sleepers laid upon or bedded in the prepared foundation. For the immersed portion of the way, it has been found convenient to construct short lengths of a timber platform upon which the rails are laid, and to float these out successively into position between guide piles, D (fig. 460). The platform has then been lowered into position by means of a winch, the necessary weight for effecting this being supplied by the ballast.*

Great care is requisite in laying the rails to see that there are no inaccuracies in the joints. In order to ensure an even bearing, it is advisable to bed the rails upon a layer of tarred felt.

The **Cradle** is a framework of timber or iron, usually consisting of three main longitudinals, of which the centre one, carrying the keel blocks, is much stronger than the other two. All three longitudinals are connected by transverse pieces of iron or wood. The latter also serve to carry the sliding bilge blocks. The whole structure is mounted over numerous cast-iron rollers, working in carriages of the same metal. Pawls are attached to the centre longitudinal of the cradle, at intervals of about 20 feet, and these engage in the rack in the permanent way and prevent any back slip. It will generally be found useful to provide short supplementary lengths of cradle to attach to the main one, in case a very large vessel has to be accommodated. A wrought-iron plough, for the removal of silt accumulations upon the rails, is a serviceable adjunct to each longitudinal.

With the object of utilising a slipway to its fullest extent, various contrivances have been adopted for releasing the cradle from its first load, in order that it may return for a second. One method of achieving this result is that of pivoting the cross pieces to the side longitudinals, so that they may be swung round to rest upon the latter. After the vessel has been drawn up to its assigned position, it is wedged up on fresh blocks placed upon the ways between the longitudinals, the cross pieces are swung round, the bilge blocks and keel blocks released, and the cradle is available for a second journey. Another method (Thompson and Cooper's) is to employ two cradles with ways constructed at different inclinations. When the vessel has reached a certain point, it is transferred from the first cradle to the second by means of fresh bilge blocks on the latter. The cradles move simultaneously, and the steeper slope of the second causes it to gradually raise the vessel off its previous bearings. In this case also the cross pieces are pivotted for removal.

Hauling Machinery.—The subject of hauling machinery will be more appropriately considered under the head of Working Equipment in Chap. xii.

* *Min. Proc. Inst. C.E.*, vol. lxxii., p. 168.

Sliding Slipways.—A distinct system of slipway from the foregoing is the sliding slipway, in which a sledge takes the place of a cradle. The ways are necessarily well greased, but in any case, the friction is greater and the wear of the structure much more considerable. The system is only adopted in isolated instances and under special circumstances, notably at Palermo,* where the configuration of the ground is precipitous. The space available was enough to admit of a slide, but not of a line of rails, the incline of which would have to be far less steep and therefore proportionately longer. The way is formed of a large number of cross sleepers on which four strong beams are placed longitudinally. Above the water level these are fixed, but the lower part is connected by hinges, and floats as soon as the weight is taken off.

Broadside Slipway.—A *Railway des transatlantiques* at Lormont, Bordeaux, has the peculiarity of withdrawing vessels from the water broadside-on, as against the general practice of taking them end-on. The slipway is 400 feet long, the length of cradle being 393 feet. Either one vessel of 410 feet length can be accommodated, or two single vessels, 203 and 180 feet long respectively. The lifting power is 3,000 tons. The extreme width is 46 feet.

The slipways at the shipbuilding yard of the Imperial and Royal Danube Steam Navigation Co., at Alt-Ofen, in Hungary, have been constructed on the same plan. They have a river-side length of 650 feet, and a breadth inland of 280 feet, of which only 180 feet is permanent way. The largest vessels accommodated are 250 feet long and 460 tons light displacement.

Stresses in Slipways.—The power required to raise a ship upon a slipway is divisible into two portions—viz. (1) that for lifting the dead weight of the vessel and its cradle, and (2) that for overcoming friction. Theoretically, the force necessary to draw a given load, W, up a smooth incline is something in excess of W sin θ, where θ is the angle which the incline makes with the horizontal. But as θ is very small in slipways, and tan θ is a much simpler quantity to deal with, the expression may be written W tan θ, without sensible error. Now, W is compounded of three items—the weight of the vessel (w_1), the weight of the cradle (w_2), and the weight of the hauling chain and rods (w_3). Of these, at least two, and sometimes all three, contribute some frictional resistance to movement, in addition to their own weight. There is the friction of the cradle rollers, and possibly that of the rods, upon the ways; and furthermore, there will be a certain amount of friction in the hauling apparatus itself. Calling the former amount f_1, and the latter f_2, and assuming a rigid base, we have the following general expression for the pull on the hauling chain :—

$$P = (w + w_1 + w_2) \tan \theta + f_1 + f_2. \qquad (132)$$

In an experiment made at the Dover slipway, where the gradient is 1 in 18, with a total load of 242 tons, it was found that the effective pull

* *Min. Proc. Inst. C.E.*, vol. xlviii., p. 297.

on the draw-chain amounted to 22·88 tons. The power absorbed in lifting was $\frac{242}{18} = 13\cdot44$ tons, leaving 9·44 tons for the power absorbed in overcoming friction. This is equivalent to 3·9 per cent. of the weight lifted.

In another experiment, made at a slipway on the River Hooghly, with a gradient of 1 in 24, the weight of the vessel and cradle amounted to 602 tons, and the effective haulage to 45·2 tons. The power absorbed in lifting being $\frac{602}{24} = 25\cdot1$ tons, this left 20·1 tons as the power absorbed by friction, or 3·33 per cent. of the weight lifted.

At Palermo the friction of a sliding slip on a gradient of 1 in 13·3 has been determined to be about 7½ per cent., and the power required, 20 per cent. of the whole load.

In Messrs. Lightfoot and Thomson's system, a ram for the return stroke has to be pushed home simultaneously with the lifting of the cradle. Indicating the pressure on this ram by the letter s, the inventors have deduced the following empirical formula from a number of actual experiments, and it has been found to answer with fair accuracy for slipways of about 1 in 20—

$$P = s + \frac{w_1 + w_2 + w_3}{8}. \qquad (133)$$

A great deal depends upon the efficiency and condition of the ways. Unless kept clean, silt and other accumulations will cause a large increase in the amount of resistance to movement. The fact also must not be overlooked that some additional force will be required to overcome the initial inertia of the load.

The Design of Graving Docks.

The principles affecting the design of graving docks do not materially differ from those enunciated in Chap. vi. for the design of entrance locks. The one exception is in regard to the floor. Locks, although the water they contain is constantly undergoing changes of level, rarely have their floors uncovered, and then only for purposes of repair. On the other hand, the very function of a graving dock demands that, for the greater portion of its useful time, it should be entirely free from water. With a natural foundation of hard and impervious rock, this fact entails no difference in the construction of the two chambers, but where the substrata are water-bearing, it is obvious that the floor of a graving dock must be made sufficiently strong to resist a hydrostatic pressure on the underside, equivalent to the greatest head of water in the immediate neighbourhood.

At first sight it may appear that, under these conditions, the graving dock floor is a beam, supported at each end by the side walls and loaded uniformly. That such is not the case is evident from the fact that few docks (if any) in the world would be capable of sustaining the estimated

load. For example, the coefficient of the breaking weight of 8 to 1 concrete, uniformly loaded, may be taken at 10 tons for a unit beam (1 foot square section and 1 foot between supports). The intensity of water pressure due to a moderate head of, say, 35 feet is 1 ton per square foot. This means for a graving dock with only 60 feet width at floor level, a uniformly distributed load of 60 tons on the underside of the floor. To adequately sustain such a load in this way, the floor would need to be 38 feet thick.

For, the breaking weight of the beam is calculable from the following formula:—

$$\text{B W} = \frac{b\,d^2}{\text{L}} \times \text{constant},$$

and substituting the known values, with unit breadth,

$$60 = \frac{d^2}{60} \times 10,$$

whence $$d = 19,$$

and taking a factor of safety of 4, which is equivalent to twice ($\sqrt{4}$) the depth, the thickness of the floor becomes 38 feet. And this is for docks of the smallest class. For docks 80 feet and upwards in width, the thickness would be even more absurdly excessive.

One simple consideration will dispose of the beam theory. There is hydrostatic pressure against the vertical faces of each extremity of the floor amply sufficient to neutralise any tension in the latter, and subject it entirely to a compressive stress. In other words, the floor must be treated as an arch, either actual or, in the case of flat floors, virtual.

If we take a permissible compressive stress for concrete of 20 tons per square foot, and consider the real or imaginary arch to have a depth or thickness of 5 feet, the rise of the invert between the centre and sides of the dock is given by a slight modification of formula (90), explained in Chap. x.

$$r = \frac{\text{W}\,l}{8\,t},$$

where r is the rise, l is the span, W $(=w\,l)$ the total weight, and t the horizontal thrust at the centre. With unit breadth $t = 20 \times 5 = 100$ tons, and

$$r = \frac{60 \times 60}{8 \times 100} = 4\tfrac{1}{2} \text{ feet},$$

and as this is to be measured to the centre line of the thickness of the arch, a flat floor only requires a maximum depth of 7 feet or so, which is a much more reasonable figure, and one which accords with results gained by experience.

One practical observation is deducible from this conclusion—viz., that the joints in masonry floors, if flat, should radiate towards the imaginary centre of the invert. It is assumed that concrete floors will be constructed in a homogeneous mass, without joints.

It has already been remarked that, where the natural foundation is sound hard rock, the necessity for an artificial floor to withstand hydrostatic pressure disappears. At the same time, care must be taken to see that there is no possible infiltration of water under the sill. The slightest film can transmit all the pressure of the external head. To prevent any such contingency—one inevitably producing disaster—it will be well to have numerous ground drains communicating with the surface of the floor, so that the water may have free vent, and the worst effect of infiltration will be some leakage or a possible flooding.

There is yet another side to the question. When a ship is dry-docked, her weight is transmitted through the keel-blocks to the floor, the centre line of which consequently undergoes a shear on each side of the blocks equivalent to this weight. And, as the imposed stress due to the vessel is best taken in the form of compression, it will be advisable to design the floor so that it may possess a second real or virtual arch, in this case upright, not inverted. A slight camber in the upper surface is useful for draining the water to the side channels.

The Construction of Graving Docks.

Masonry at one time constituted the material most in favour for the construction of graving docks, but of late years Portland-cement concrete has superseded it to a very large extent. Either material is extremely durable,* but concrete has the advantage of greater economy in most cases. Timber has been, and is, largely used in the United States. It is, however, much inferior to stone or concrete in durability, and there are indications that the desirability of a more permanent form of construction is becoming recognised. One advantage of wood is stated to be that it is safer than stone to work upon in frosty weather, ice being less likely to form and remain upon its surface. The claim is of dubious validity. Another contention, that timber-work is injured less than masonry by the severity of North American winters, strikes one as being untenable and even absurd, if any analogy exist in the behaviour of the two materials in this country. Timber docks are certainly much cheaper to construct, and herein, apparently, lies their most effective recommendation.

The methods adopted in building stone or concrete docks are identical with those in building locks, and the general features of these having already been discussed in Chapter vi., the subject need not be further considered.

In the construction of timber docks, the most prevalent practice is as follows:—The site of the floor is first enclosed within continuous sheet piling, formed of half-timbers having tongued and grooved joints, and the whole area is then studded with bearing piles of whole timber, driven at

* The deterioration of concrete work in certain graving docks, as at Belfast and Aberdeen, has been the subject of an inquiry in the Chapter on "Materials," to which the reader is referred for an explanation of the phenomena.

intervals of 3 feet or more. Under the keel-blocks, the piling is still more concentrated. Their heads having been cut off to a uniform level, the piles are connected by longitudinal and transverse beams some 12 inches square, upon which is laid the 3-inch planking forming the floor surface. The pile-tops and the longitudinals are bedded in concrete provided with a smooth sloping surface to drain off the water.

Square balks, set at an angle of about 40 degrees, form an inclined foundation for the altar courses at the sides of the dock. The courses have vertical and horizontal faces and splayed undersides. The supporting timbers are carried on rows of piles, pitched about a yard apart. Above the concrete under floor level, the sides are backed with clay-puddle, confined within a second and outer row of sheet-piling driven well down below the floor level.

A combination of timber, stone, and concrete construction is exemplified at a graving dock (fig. 461) at Halifax, Nova Scotia.* The substratum of the dock is rock, and it was proposed to form a floor of concrete upon this

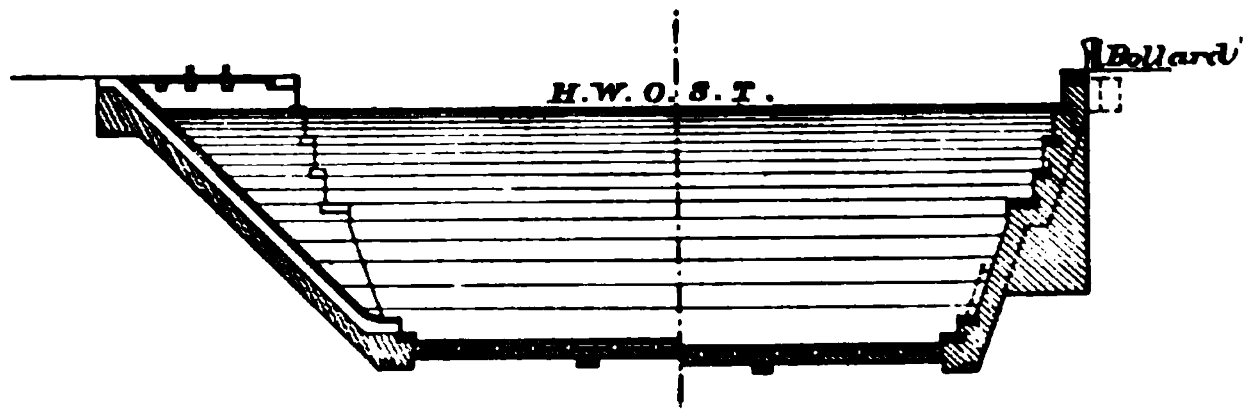


Fig. 461.—Section of Graving Dock at Halifax, Nova Scotia.
Scale, 50 feet = 1 inch.

foundation, 2 feet thick, but, to meet the wishes of the City Corporation, a pitchpine floor was substituted, laid on sleepers bolted to the concrete, which was reduced to a minimum thickness of 12 inches. "The flooring has proved a great convenience, as, when the dock is pumped out, the water drains away from the surface immediately." Above the rock level the walls consist of rubble-in-cement backing, and they are faced with concrete, 3 feet thick, from top to bottom, the altars being capped with granite, 12 inches thick.

Mud docks of a very primitive description are apparently still in use at some insignificant oriental ports, but they do not call for any serious notice.

Types of Floating Docks.

The earliest floating docks were of wood, with a sectional profile resembling that of a ship. They were fitted with a pair of gates at one end, which were closed after the entrance of a vessel, and the impounded water was then pumped out.

Wooden docks of a later type are known as the "Sectional Dock" and the "Balance Dock."

* Parsons on "Halifax Graving Dock, N.S.," *Min. Proc. Inst. C.E.*, vol. cxi.

"The **Sectional Dock**, as its name implies, is divided into as many sections as are required for the particular vessel to be docked. Each section consists of a rectangular wooden box made watertight, and in the ends of these there is an open wooden framework of a height somewhat greater than the depth to which it is proposed to sink the dock. Within this frame a wooden watertight box slides up and down, which can be fixed by means of a rack and pall to any required position. These boxes or tanks serve the purpose of keeping the base or lower part of the dock steady, water not being allowed to enter therein. Thus, a complete dock consists of a series of eight or ten independent compartments below, with two movable air chambers to each; and, although there are certain timbers connecting the different boxes, they are not constructed so as to enable any box to support the adjoining ones." *

A disaster occurred to one of these docks at Callao, involving the sinking of a ship, by reason of the disconnected compartments yielding to unequal stress.

TRANSVERSE SECTION.

Fig. 462.—Cartagena Floating Dock.

The **Balance Dock** is an attempt at improving the sectional dock. There is only one compartment, subdivided internally into a series of separate chambers. Docks constructed on this principle have generally been successful, and have had considerable vogue in the United States.

Iron docks came in about the year 1859. They were introduced by Mr. Rennie at the naval arsenal at Cartagena, and based on the principle of the balance dock of Mr. Gilbert. A transverse section of the Cartagena Dock is shown in fig. 462. It had a length of 324 feet, a breadth of 105 feet, and a lifting power of 11,500 tons.

In 1860, the elliptical, or **U** section, which had disappeared with the earliest timber types, was reproduced in the Bermuda Dock, and continued to be adopted at intervals. At the present time, the rectangular shape is almost the invariable rule. The **U** section was generally fitted with gates

* Rennie on "Floating Docks," *Min. Proc. Inst. C.E.*, vol. xxxi.

to increase its stability; gates and caissons are quite unnecessary and are rarely used in connection with the rectangular section.

Herr Howaldt, of Kiel, advocates a system of composite docks which he has devised, the frames being of iron or steel and the deck and bottom sides of wood. He states, as the result of his experience, that while with metal plating, the girders must not be more than 2 feet apart, with planks of pitchpine or beech, 4 inches thick, the frames can be placed 4 feet apart, without the least deflection in the panels. The advantages claimed for the system are economy in construction and maintenance (wood requiring less attention than iron), and a certain amount of natural flotation, which reduces the pumping power required. This last contention is of doubtful value: the bulk of a wooden ship largely discounts its natural flotation.

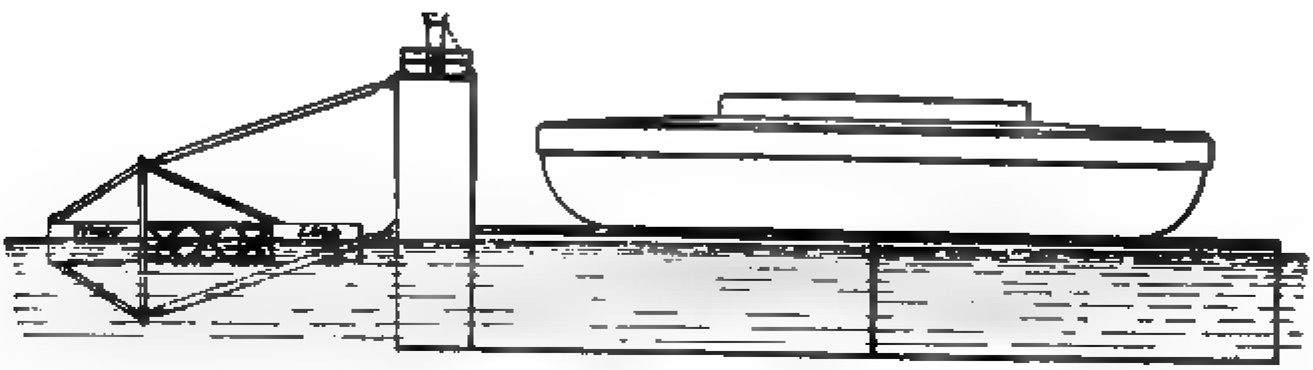


Fig. 463.—Depositing Dock.

Fig. 464.—Off-shore Dock.

The restriction in beam-accommodation imposed upon a double-sided dock led Messrs. Clark and Standfield, about the year 1877, to design the depositing dock, in which one of the vertical sides is suppressed. This has given rise to two varieties, according to the means adopted for maintaining equilibrium. The term **Depositing Dock** (fig. 463) is applied to a dock freely floating and balanced by an outrigger. A similar dock connected with the shore by means of hinged arms attached to strong columns, is known as the **Off-shore Dock** (fig. 464). The off-shore dock is much more

stable than the depositing dock, though the latter is, of course, well within the limits of practical safety. Another difference between the two types is that the off-shore dock is constructed in one continuous pontoon, which is a lighter form of construction than the separate caissons of the depositing dock.

Design of Floating Docks.

The design of floating structures being the particular province of the naval architect, it is manifestly outside the range of the present work to enter into a discussion of any of the specific problems or details connected with the disposition and arrangement of floating docks. The broad principles of the equilibrium of floating bodies have already been enunciated, in connection with the subject of dock caissons, in Chapter viii., and it would certainly be inadvisable to do more than supplement the information therein contained by a few remarks of a general nature, relating to the subject at present under consideration.

In the first place, then, with a given length and displacement, an increase in breadth means an increase in stability, hence a broad beam is an advantage to a floating dock. The usual proportion of beam to draught lies between 8 and 10 to 1.

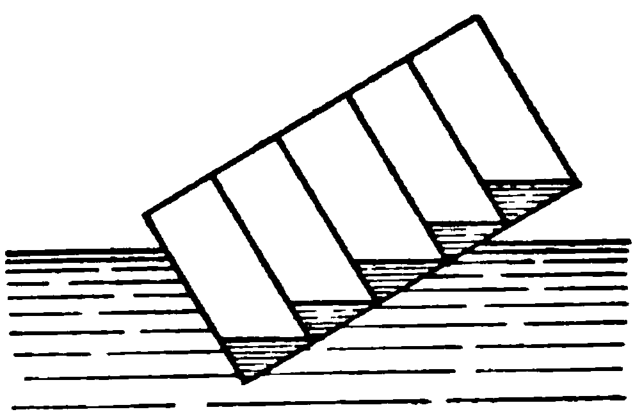


Fig. 465.

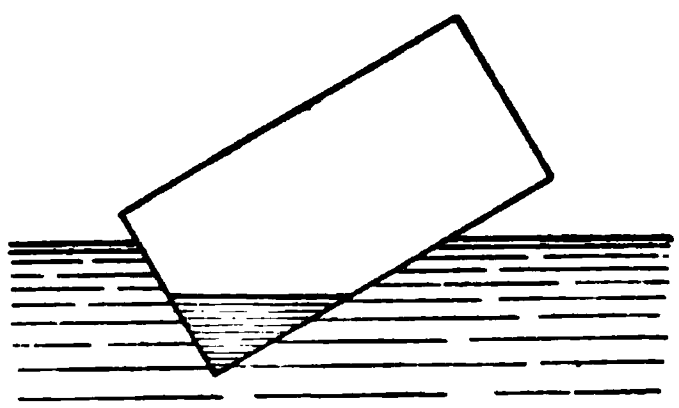


Fig. 466.

Secondly, a rectangular transverse section has a lower centre of gravity than a curved section, and therefore is more stable. It has already been pointed out that docks with elliptical or U-shaped profiles were fitted with gates; the object of these is to lower the centre of gravity.

Thirdly, the less the height of the sides, consistent with the requirements of shoring, the less tendency there will be to top-heaviness, with the concurrent advantages of greater light and ventilation in dealing with vessels to be repaired.

Lastly, the more compartments in a cross-section, the greater the stability under water ballast. This is evident from figs. 465 and 466, which represent two floating tanks containing water, and slightly displaced. The distribution of the water after displacement is much more uniform in the subdivided tank than in the other, and there is also less surging motion.

Process of Overhauling a Self-Docking Floating Dock.

The manner in which a modern floating dock, constructed on Messrs. Clark and Standfield's system, undergoes a thorough overhauling is both ingenious and interesting. It will be readily understood from an inspection of figs. 467 to 474.

In the first operation the vertical sides are dealt with. The whole structure is sunk, to the extent shown in fig. 467, by admitting water to the side and floor compartments alike. The dock is then raised by

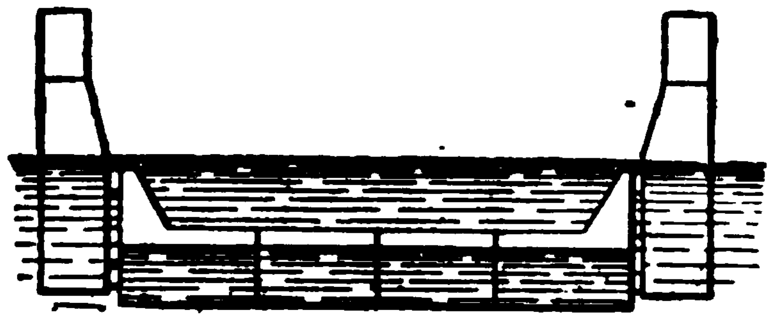


Fig. 467.

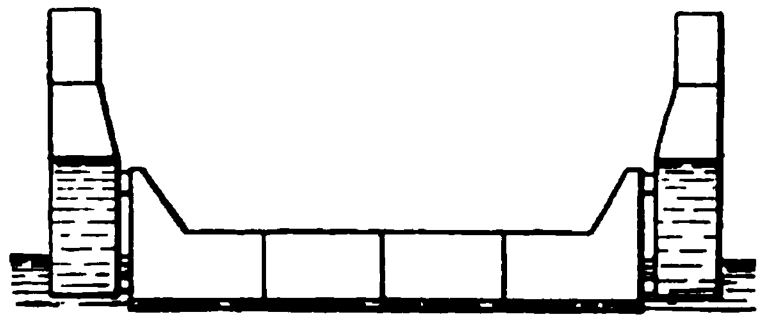


Fig. 468.

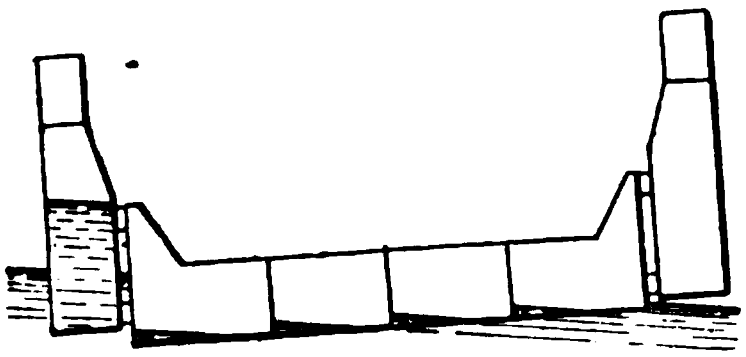


Fig. 469.

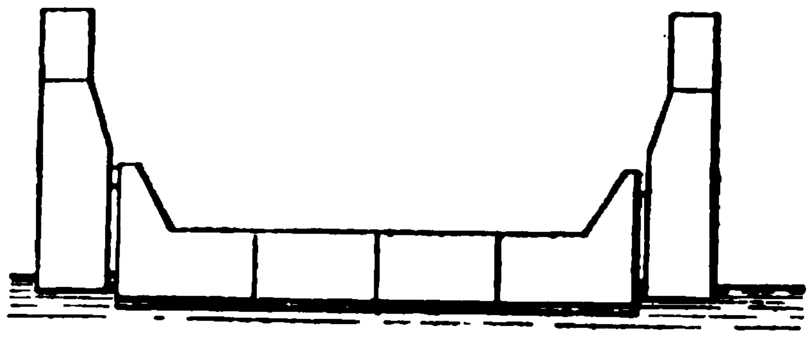


Fig. 470.

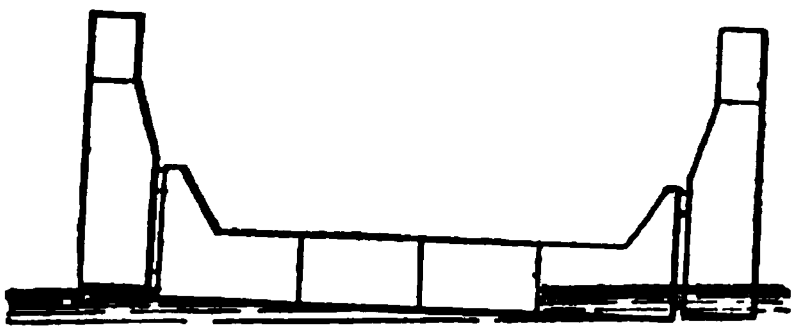


Fig. 471.

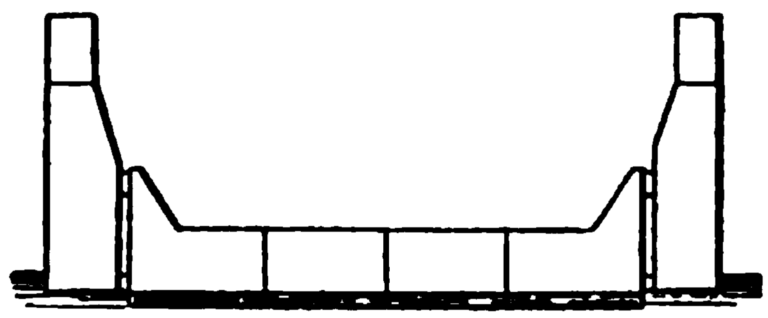


Fig. 472.

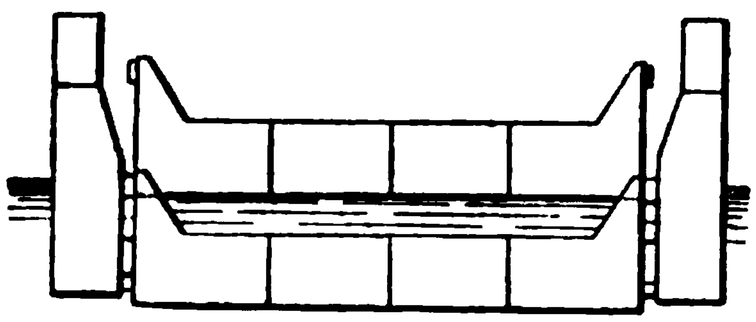


Fig. 473.

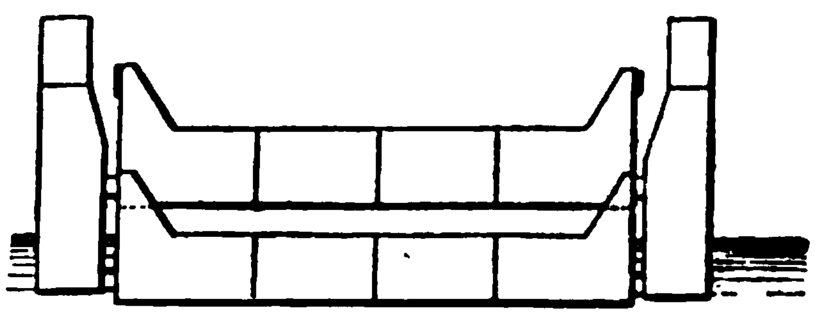


Fig. 474.

pumping out the floor pontoons alone. This brings it into the position shown in fig. 468. Next, the water is allowed to escape from one of the sides, and the dock takes a list sufficient to raise that side well out of the water (fig. 469). Each side is dealt with in this manner. The dock is then restored to level trim, with all its compartments empty (fig. 470).

The dock floor is next undertaken. It is formed of a series of pontoons extending the whole internal width of the dock, and capable of being

entirely disconnected from each other and the sides. Usually one pontoon is dealt with at a time, and the operation consists in raising it above the rest. The pontoon is separated from the sides by an open space of 2 feet, to which water has free access. Attachment is made by means of "fish-plate" joints, consisting of steel lugs, secured together by steel taper pins. The drainage junction-pipes between the pontoon chambers, and the pumps in the side walls, are first of all disconnected. Then the taper pins are withdrawn. To complete this step, it is necessary to tilt the dock slightly, as in fig. 471, even trim being afterwards restored (fig. 472). One pontoon is now floating clear of the remainder of the structure. The dock is next sunk by the re-admission of water into its compartments until the relative positions are as shown in fig. 473. At this stage the floating pontoon is re-connected to the sides, at a higher level, by means of similar lugs and pins. The dock is raised bodily by pumping, the single pontoon leaves the water, and the operation is complete (fig. 474).

Docks of L section have their component pontoons berthed upon one another, in the ordinary manner of docking a vessel.

Equipment of Repairing Docks.

The various items for the equipment of a repairing dock include keel-blocks, bilge-blocks, side-shores, lifting cranes, capstans, snatch blocks, bollards, hooks, &c.

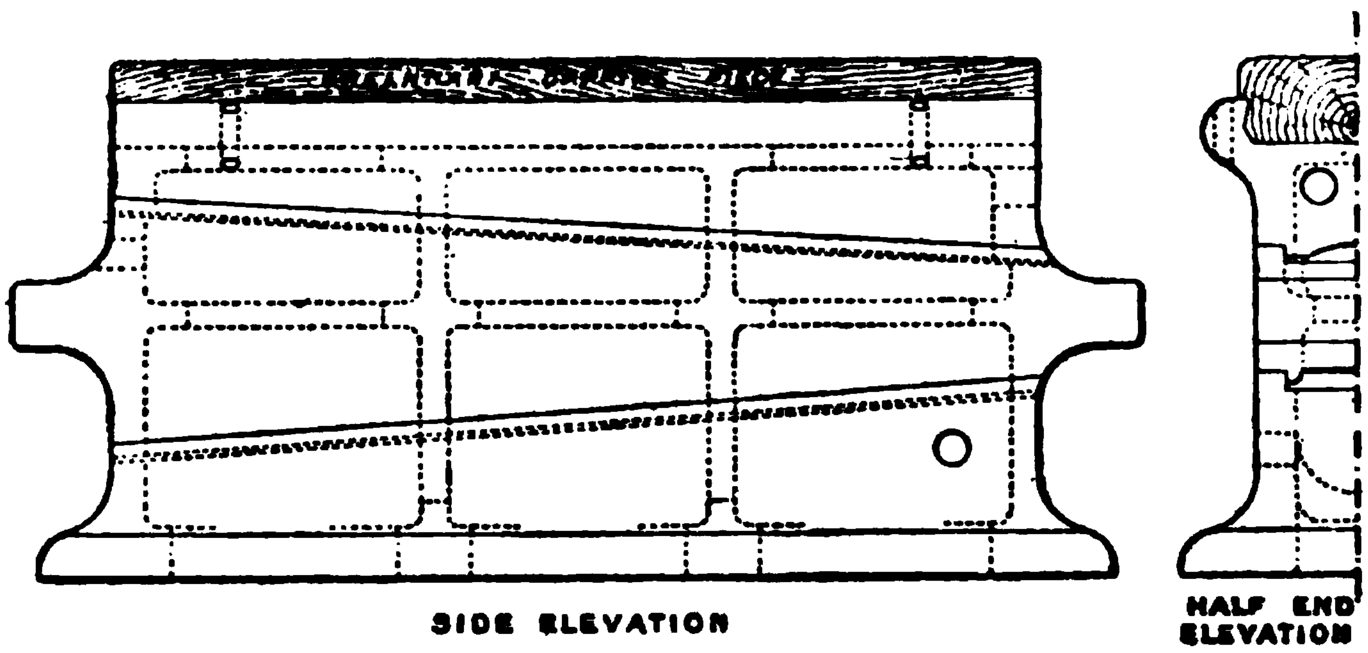


Fig. 475.—Keel-block, Belfast.

1. **Keel-blocks.**—These are for the purpose of affording a uniform and level base for a ship's keel, and in order to give ready access thereto, they stand a few feet above the dock floor. The height usually ranges between 2 feet 6 inches and 4 feet. The greater height involves a corresponding additional depth of dock to accommodate the same class of vessel, but owing to the headroom it affords, the cost of repairs is reduced. The best material for keel-blocks is a matter of dispute. Cast iron was very largely employed, with timber caps, until the accident to the "Fulda" threw cast-iron blocks into disrepute; most unjustifiably, because accidents have occurred with

other kinds of blocks. Many engineers prefer wooden blocks—oak for preference, pitchpine often on account of its cheapness. In the latest and largest graving dock at Liverpool the blocks are of cast iron surmounted by a 12-inch birch log, capped with 3 inches of soft wood. Similar blocks, capped with greenheart, are used at Belfast (fig. 475). On account of the flotation, wood blocks must be anchored. Curved cast-iron caps have been used at Amsterdam, the object being self-adjustment between block and ship. With the same object in view, hydraulic blocks have been proposed, but the consequent uniformity of pressure produced all the effects of rigidity, and the method was abandoned after trial. Steel, greenheart, elm, and teak have also been employed for blocks.

The distance apart of the block centres varies from 2 to 5 feet, being governed by the load to be carried. Wide intervals, where possible, are convenient. On the other hand, it is often necessary to support a large ship by inserting temporary intermediate blocks between the permanent ones. The large Atlantic liners are continuous-blocked in this fashion for a great portion of their lengths.

As regards shape, wedge-shaped blocks have been found most convenient for adjustment. The wedges should be readily removable and portable. Wooden blocks, however, are generally rectangular and bound at the ends with iron bands.

Bilge-blocks or Side Cradles are not so commonly employed in graving as in floating docks, though they form useful adjuncts to keel-blocks. Their drawback is that they rather interfere with freedom of movement, and consequently they are arranged at greater intervals—say, about 50 feet. Their upper surfaces have to be adjusted to the level of the ship's bilge. Sometimes props under the bilge keels are substituted for them.

Side-shores.—These form a series of lateral supports to a vessel upon the blocks. They are of wood, about 9 or 10 inches square at the centre, tapering slightly each way to the ends, which are bound with iron. They are lowered into position as the pumping proceeds, in somewhat primitive fashion by means of ropes, and are tightened up with wedges, so that one end bears firmly against the vessel's side and the other against an altar course. It has been suggested that a series of horizontal steel shores, worked in and out of the sides of the dock by mechanical means, would be a great improvement. No doubt the method would be more scientific, but it has certain obvious difficulties attached to it in the way of regulating the level of the shores so as to suit ships of all sizes. Moreover, since pumping is a process involving some time, ample opportunity is afforded for setting shores by hand without causing extra or undue delay.

Hooks, fixed to the quay at coping level with rope moorings, are sometimes employed for securing a vessel in position, more especially when she is only shored on one side, as in the case of a graving dock capable of accommodating two ships side by side. The vessel is then given a slight list towards the nearer quay. Bollards and mooring posts serve the same purpose.

Timber Slides, of smooth granite, in the side walls are handy for the purpose of lowering shores and other timber, but their use is not universal, as in many cases the logs and planks required are thrown into the water while pumping is still proceeding.

A **Rudder Pit** is a useful feature in the event of the removal of a ship's rudder, though many graving docks are without them, and they are only required on rare occasions. There are two such pits at the Canada Graving Dock, Liverpool, each 50 feet long, 6 feet wide, and 16 feet 6 inches below floor level. When they are not in use for this particular purpose, the keel-blocks are continued over them, being supported on stout girders.

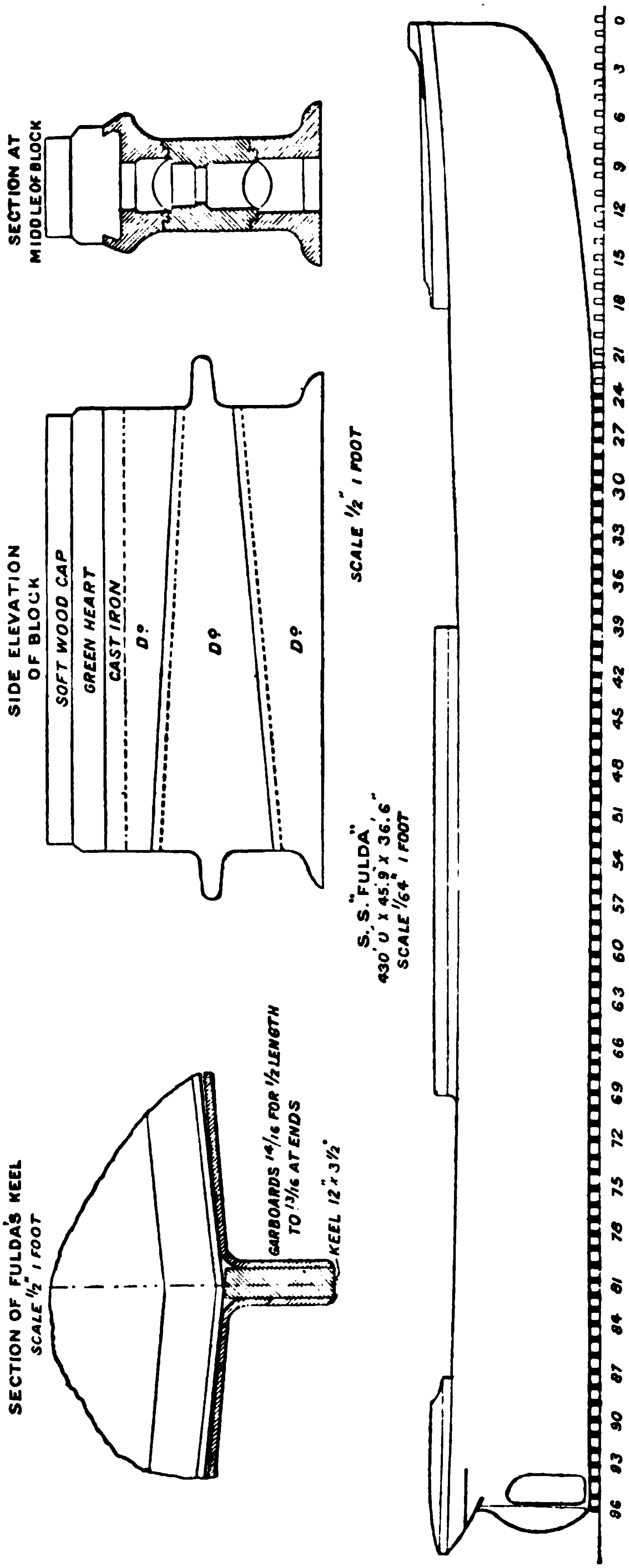
A **Travelling Crane** of large power and wide range, for lifting heavy machinery in and out of a vessel, is essential. A propeller may have to be lifted from the dock bottom or even from the ship's hold, and in the latter case it would have to clear the hatchway coaming and the bulwark, both of which will in most cases be above the coping level. A lifting power of not less than 25 tons and up to 50 tons should be provided, with a clear outreach beyond the centre line of dock floor, and a height of 30 feet from coping level to under side of jib. The great amount of outreach is more particularly requisite in the case of vessels with twin screw propellers.

In addition to hydraulic mains for gate machinery and cranes, it is an advantage to utilise the pipe trenches for the conveyance of electric or pneumatic power to drilling machines, which are very commonly needed to remove plates from the hulls of ships. Portable electric lights are extremely serviceable beneath a ship's hull.

Distribution of Pressure on Keel-blocks.

The distribution of the pressure of a ship's keel over the blocks in a graving dock is a very difficult and complex problem, and one to which no satisfactory solution has yet been propounded, despite the attention which it has received from many eminent scientists and technical experts. Yet the question cannot be ignored, for it has been the cause of several accidents of a very serious nature. A recent disaster which has attracted widespread notice and caused much misgiving, if not dismay, is that which occurred to the North German Lloyd s.s. "Fulda" while in No. 2 Graving Dock, Birkenhead, on 2nd February, 1899. Not more than 15 or 20 minutes elapsed, from the time she was left dry upon the blocks till she crashed through them to the dock floor and received such injuries as to become a total loss, constructively.

The data involved are these:—The "Fulda" was a vessel 430 feet in length between perpendiculars, 45 feet 9 inches in breadth, moulded, and 36 feet 6 inches in depth, moulded. Her displacement, as laden at the time of the accident, was about 6,600 tons; she had a bar keel 12 inches deep and 3½ inches wide; the blocks upon which she was docked were of cast iron, with 6-inch greenheart caps and 3 inches soft wood on top of the greenheart (figs. 476 to 479). These blocks were 2 feet 6 inches high and



Figs. 476, 477, 478, and 479.—The s.s. "Fulda" and Graving Blocks.

4 feet 6 inches apart from centre to centre. The amount of overhang forward was very great. At one-fourth of the vessel's length, measured from the stem, the keel rose $\frac{3}{4}$ inch and continued to rise as it proceeded forward. The condition of things is shown in fig. 479, the black blocks indicating the extent of the supported part of the keel.

The exact sequence of the occurrence was never clearly ascertained, the evidence being somewhat conflicting, but the most competent witness stated that he found the blocks flying out at the after end of the ship first, and then the forward blocks came down. On the other hand, Dr. Elgar, who was called in as a consulting expert by the Mersey Docks and Harbour Board, inclined to the opinion that the catastrophe was due to the great pressure imposed on the forward blocks by the excessive overhang of the vessel's stem, and, therefore, that disruption began in that quarter.

In a paper* read before the Institution of Naval Architects the same year, Dr. Elgar gave the reasons for his decision, and entered very minutely into the question of the probable amount of stress produced in the foremost loaded block, deducing a pressure of $178\frac{1}{3}\frac{4}{7}$ tons. Without desiring in the least to depreciate the care and skill with which the mathematical calculations were carried to their conclusion, it cannot but be felt that the postulates were too hypothetical to justify any definite numerical result. It is, in fact, only possible to approach the question by means of certain assumptions, none of which may be accurately, or even approximately, true. For example, it has to be taken for granted, either that the blocks were elastic or that they were rigid, the keel flexible or inflexible, and the probability is that no one of these conditions prevailed throughout.

It is useless to go into the matter again in so far as the "Fulda" is concerned. Whether the blocks were sheared at the forefoot or abaft the middle (and it is a strange complication of the whole affair that the blocks had been in use for 40 years and the "Fulda" docked several times before without mishap), the fact remains that the pressure upon the keel-blocks is very unevenly distributed, and is certainly very great on the forward blocks under any ship of ordinary design.

About the time of the "Fulda" disaster, the author made a number of careful observations of the actual profile assumed by the keels of various vessels in graving dock. In all cases he found two regions of great depression—one immediately abaft the forefoot and the other amidships under the machinery. In these localities, the keels had crushed the soft wood caps to a much greater extent than elsewhere, there being a maximum difference of level in some cases of as much as $1\frac{1}{2}$ inches.

The intricacies of the problem are too numerous for any exact solution, but if we choose to confine our investigation within certain limits, we may arrive at a result which will have some relative value. We will therefore briefly deal with the general case of the distribution of stress under a system of irregular loading, making the following assumptions:—

* Elgar on "The Supporting of Ships in Dry Docks," *Min. Proc. Inst. N.A.*, 1899.

1. That the vessel is a rigid structure—*i.e.*, there is no bending or yielding in any part of the keel.

2. That the blocks are perfectly elastic—*i.e.*, the amount of compression is proportional to the load.

3. That the line of keel coincides with the line of blocks—*i.e.*, there is no initial stress due to a cambered keel.

The converse of all these postulates is equally likely to hold good in practice.

Let A B = l, represent the total length of a ship (fig. 480), and O R, the vertical line through its centre of gravity, taken for simplicity through the centre of the ship. Let W be the total weight.

If now the vessel be supported uniformly throughout its entire length, the pressure diagram will be the rectangle A B C D, in which if A D = a, and A B = l, then $a\,l$ = W.

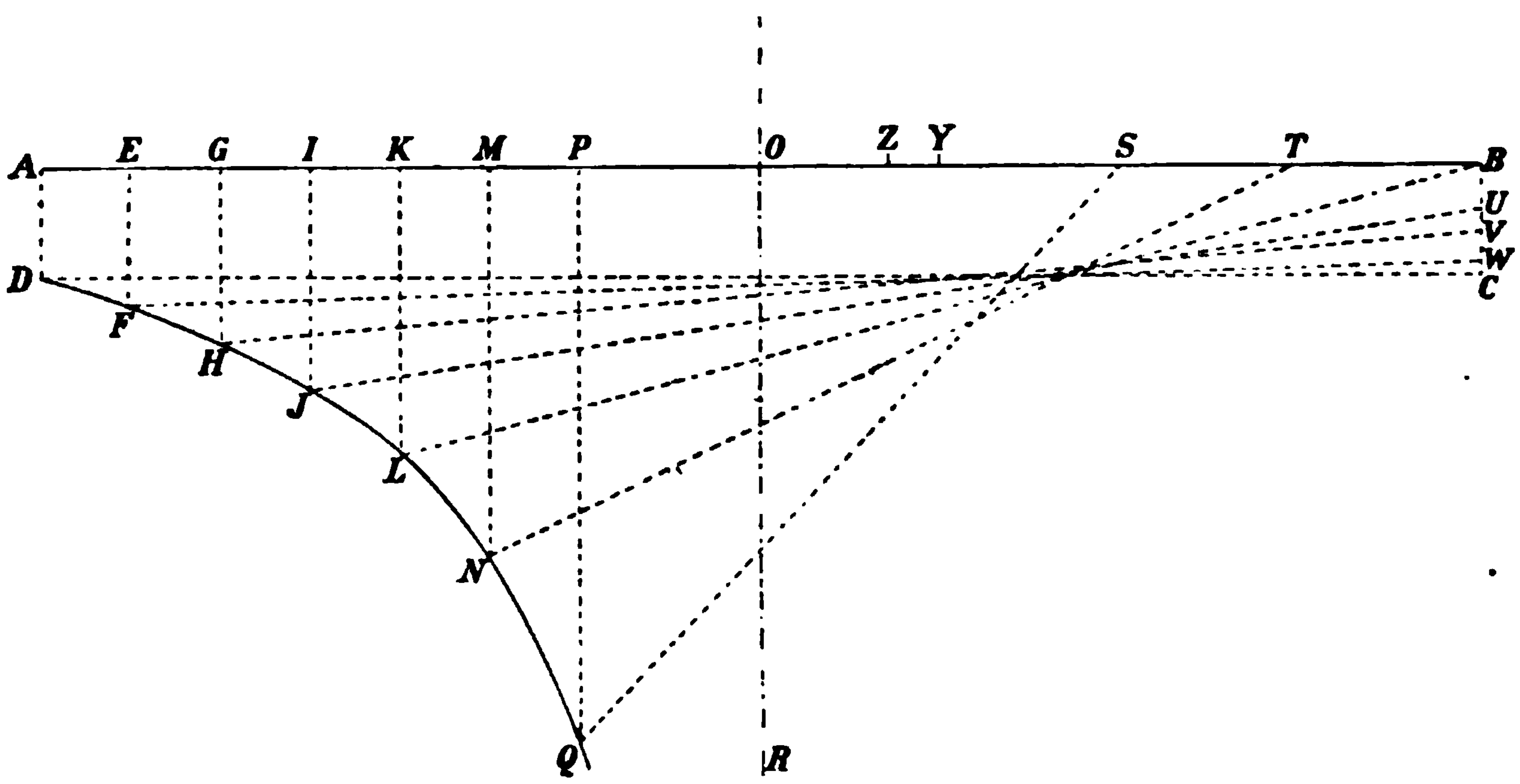


Fig. 480.

Secondly, suppose the vessel to have an overhang equivalent to one-fourth of her length, so that the supported portion of her keel is K B = $\frac{3}{4}\,l$. If the load were distributed over the supported portion only, so that the centre of pressure coincided with the middle point, Y, of that length, there would be a uniform intensity, a_1, determined by the consideration,

$$a_1 \times \tfrac{3}{4}\,l = \text{W}. \qquad . \quad . \quad . \quad . \quad (134)$$

But this is not the case, for the centre of pressure is still at O, while the centre of support is at Y, giving an eccentricity O Y = $\frac{\text{K B}}{6}$; *i.e.*, one-sixth of the supported length. Now, we have already determined in connection with stresses in wall joints (Chap. v.) that when the eccentricity of pressure is one-sixth of the length of the base, the intensity is zero at the inner, or further, edge, and a maximum of twice the mean uniform stress at the outer edge. Hence, if we draw K L = $2a_1$, and join

L B, we have K L B as the pressure diagram for an overhang of one-fourth the vessel's length.

For overhangs greater than this, we may proceed by analogy thus:—Take the point M as the limit of the supported length, and make O T = 2 O M. Then the eccentricity of the point O is $O\,Z = \frac{M\,T}{6}$. Hence, make

$$M\,N = 2\,a_2 = \frac{2\,W}{l_2},$$

where l_2 is the length of effective base M T. Join N T, and M N T is the pressure diagram. Although the vessel apparently receives support from T to B, such, as a matter of fact, is not the case, the compressive stress passing through zero value at T to a negative value beyond that point. In other words, there would be a gradually increasing tensile stress from T to B, if the vessel were fastened down to the blocks.

Similarly, if the forefoot extend to P, take O S = 2 O P. Draw P Q = 2 a_3, where

$$a_3\,l_3 = W_1 \text{ as before.}$$

Join Q S_1, and P Q S is the pressure diagram under these conditions.

Any number of points may be found in this way, and since

$$L\,K \times K\,B = N\,M \times M\,T = Q\,P \times P\,S = 2\,W,$$

we may write down the general equation—

$$x\,y = \text{constant}, \quad . \quad . \quad . \quad . \quad . \quad . \quad (135)$$

so that the curve L N Q is a rectangular hyperbola, with its origin at O, and the lines O A and O R as its asymptotes.

This equation is only applicable to values of x not exceeding K O. When the overhang of the vessel is less than one-fourth of the total length, the compression does not vanish at B, but gradually increases as the forefoot decreases, until it attains a maximum value of B C, with the disappearance of all overhang at the stem.

Consequently, we must substitute for (135) another equation conforming to the altered conditions. We obtain this readily from the investigation in Chap. v. already alluded to. There it was seen that when the eccentricity was less than one-sixth of the base, the value of the greatest intensity of pressure at the outer, or nearer, edge of the joint was

$$Y = a + y,$$

where a = the uniform intensity due to zero eccentricity under similar conditions of load, and

$$y = \frac{6\,a\,x}{l},$$

x being the eccentricity.

Apply this to the case where the rise of the vessel's stem begins at the point I. Then the length of base is I O = l_4, and

$$a_4 = \frac{W}{l_4}.$$

The eccentricity is $\frac{l}{2} - \frac{l_4}{2} = x_4$, which is accordingly known. Make

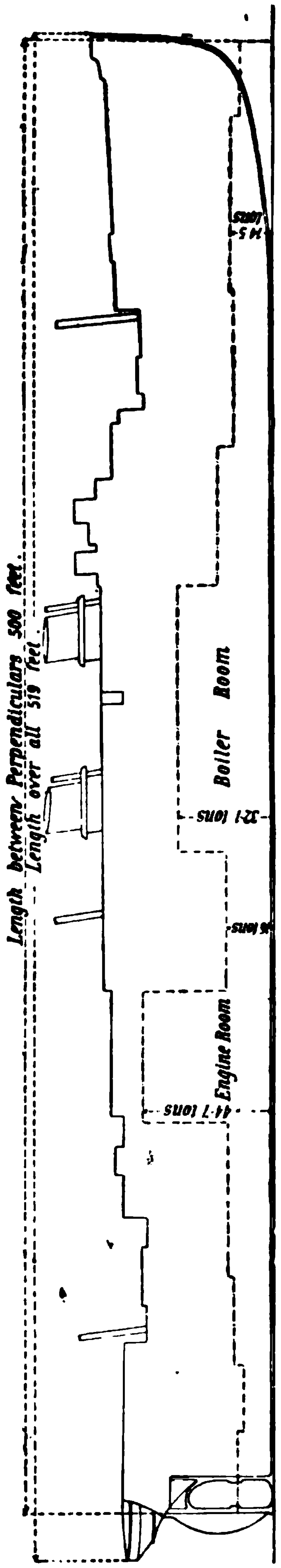


Fig. 481.—Distribution of Weight, s.s. "Umbria" and "Etruria."

$$I J = \frac{W}{l_4} + \frac{6 W x_4}{l l_4},$$

and

$$C U = \frac{W}{l_4} - \frac{6 W x_4}{l l_4}.$$

Join J U, and I J U B is the pressure diagram.

Similarly at G we have

$$a_5 = \frac{W}{l_5}, \text{ where } l_5 \text{ is the length G B;}$$

$$G H = \frac{W}{l_5} + \frac{6 W x_5}{l l_5},$$

and

$$C V = \frac{W}{l_5} - \frac{6 W x_5}{l l_5},$$

x_5 being, of course, $\frac{l}{2} - \frac{l_5}{2}$. Joining H V, we have the pressure diagram G H V B for these particular conditions.

In this way, we have found a series of points, D F H J L N Q. Joined together, they form a curve which may be termed the curve of maximum pressures, since any ordinate, E F, G H, I J, &c., is a measure of the intensity of pressure at the outermost keel-block, corresponding to an overhang from that point.

With the limited space at disposal, it is not possible to enter into a discussion of the modifications caused in the above equations by a departure from any of the conditions upon which they are based. It must be stated that such modifications will often entail values largely exceeding those given, but as they, too, are founded upon a series of hypotheses, no practical advantage would accrue from the very complicated investigation of their influence. The point to be really emphasised is the absolute necessity for a large margin of safety.

The diagram (fig. 481) shows, in the case of the Cunard s.s. "Etruria" and her sister ship the "Umbria," the relative distribution of weight throughout the length of each vessel. The dotted line indicates to scale the intensity of pressure upon the keel-blocks,

supposing each vessel to be uniformly supported throughout her entire length.

The following table illustrates the amount of overhang in typical ships of the present day :—

TABLE XXXVI.

Name of Vessel.	Extreme Length.	Overhang.		Keel.		Nett Registered Tonnage.
		Forward.	Aft.	Kind.	Size.	
	Feet.	Feet.	Feet.		Inches.	
Cevic, . . .	520	52	41	Plate,	10 × 3	5,402
Georgic, . ..	570	52	36½	,,	10 × 3	6,570
Tauric, . . .	476	52	8	,,	10 × 3	3,670
Lake Superior, .	415	25	...	Bar,	5½ × 9	2,897
Manchester City, .	461	55	45	...	...	3,727
Cymric, . .	600	52	33½	Plate,	12 × 3	8,508
Etruria, . .	520	95	...	Bar,	6 × 10	3,690
Teutonic, . .	580	78½	35	Plate,	17 × 3	4,269
Campania, . .	620	95	...	...	...	4,973
Cufic, . . .	444	42½	10	Bar,	5½ × 9	3,122
Parisian, . .	455	27½	...	...	...	3,385
Aurania, . .	488	50	10	Bar,	7 × 10	4,029
Oceanic, . .	705	87½	40	Plate,	18½ × 3½	6,916
Winefredian, .	570	55	...	,,	11 × 3	6,816
New England, .	570	55	15	,,	12 × 3	7,416
Norseman, . .	510	42½	12	,,	10 × 3	6,129
Afric, . . .	570	45	13	,,	12 × 3	7,804
Ivernia, . .	600	80	30	...	...	9,052
Celtic, . . .	704	62½	32½	Plate,	18½ × 3½	13,448

Gridirons at Liverpool.

There are two gridirons existing at Liverpool. One, in a recess at the Clarence Graving Dock Basin, has a length of 313 feet 6 inches and a breadth of 25 feet 6 inches. The logs or blocks (fig. 482) are 11 inches wide by

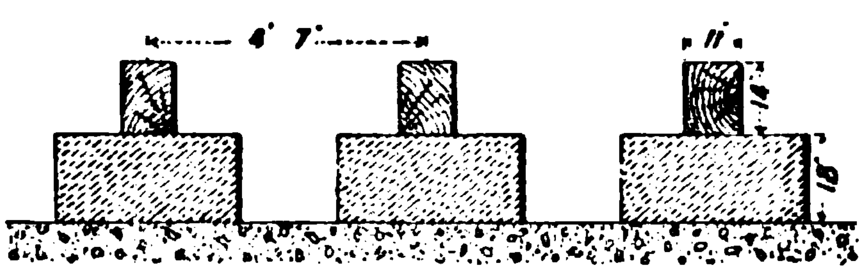


Fig. 482.—Gridiron at Liverpool.

14 inches deep, laid 4½ feet apart, centre to centre, upon masonry blocks on a concrete foundation. There is a fall of 2 feet 5 inches in the length of the gridiron, the lower end of which is 20 feet below high water of ordinary spring tides. The other gridiron is at the King's Pier, and has a length of 509 feet and a breadth of 26 feet. The blocks in this case are level throughout.

Hydraulic Lift at London.

The following account of Clark's hydraulic lift at the Victoria Docks, London, is extracted from an article by Mr. G. B. Rennie in the *Practical Mechanic's Journal Record of the Great Exhibition of 1862*:—

"The lift (fig. 483) consists of an excavated channel, of about 300 feet long and about 60 feet broad, on each side of which 16 cast-iron columns, 5 feet in diameter, are sunk about 12 feet into the ground, 20 feet from centre to centre. At the bottom of the column there is a hydraulic press or lift. The diameter of the ram is 10 inches, with a travel of about 25 feet. On the top of the piston or ram a wrought-iron crosshead is fitted, from which iron links are suspended and connected with a cast-iron girder, one on each side of the column, so that there are 16 coupled girders of about 60 feet length and 20 feet apart, each couple being suspended and lifted by

Fig. 483.—Hydraulic Lift.

two hydraulic rams or pumps. On the top of these girders a pontoon is placed at the requisite length. These pontoons vary from 150 to 320 feet in length, and are 59 feet broad. The smaller are placed on 8 sets of coupled girders and the larger on the whole 16. They are made of sufficient depth for stiffness and in order to give the required displacement, so that when empty they have buoyancy enough to keep the vessel well out of the water. The pistons or rams are worked by a pair of horizontal engines made by Messrs. Easton & Amos. These engines are on the expansive condensing principle, with one high-pressure cylinder of 23 inches diameter and 2 feet stroke, and two expansive cylinders of 33½ inches diameter with the same stroke. The steam expands from the small cylinder into the two larger ones; pressure of steam per square inch, 50 lbs.; indicated horse-power, 120. The engines work 12 hydraulic force pumps of 1·96 inches diameter and 2 feet stroke in three groups—viz., two groups of 3 and one of 6 pumps.

The amount of pressure obtained is 28 cwts. per circular inch, equal to about 4,000 lbs. per square inch. From these pumps the water is discharged through wrought-iron pipes, ½ inch internal diameter and 1 inch external, and above 10,000 feet in length.

"The following is the manner of docking a vessel:—The girders with the pontoon upon them are allowed to sink to the depth required for the particular vessel to be docked. She is then hauled over the pontoon and on to the blocks and shored, or rather wedged up by movable bilge blocks instead of breast shores. The pontoon and vessel are lifted out of the water and the water in the pontoon allowed to escape by valves. When empty the valves are closed, the girders lowered, and the pontoon left to bear the whole weight of the vessel and to be moved into any suitable position. To give greater accommodation Mr. Edwin Clark arranged a system of shallow docks, eight in number, communicating with a shallow basin of about 500 feet square, into one of which the pontoon has to be floated. The space occupied by the docks and basin is about 25 acres. Many vessels have been already lifted and repaired in this manner, the largest of which, the 'Calcutta,' is of 1,800 tons burthen."

Slipway at Dover.*

Originally constructed in 1849, the Dover slipway underwent an enlargement in 1888, being lengthened to 556 feet, with a capacity for vessels up to 850 tons deadweight. The gradient is 1 in 18, and the width at quay level, 52 feet (see figs. 484 to 491).

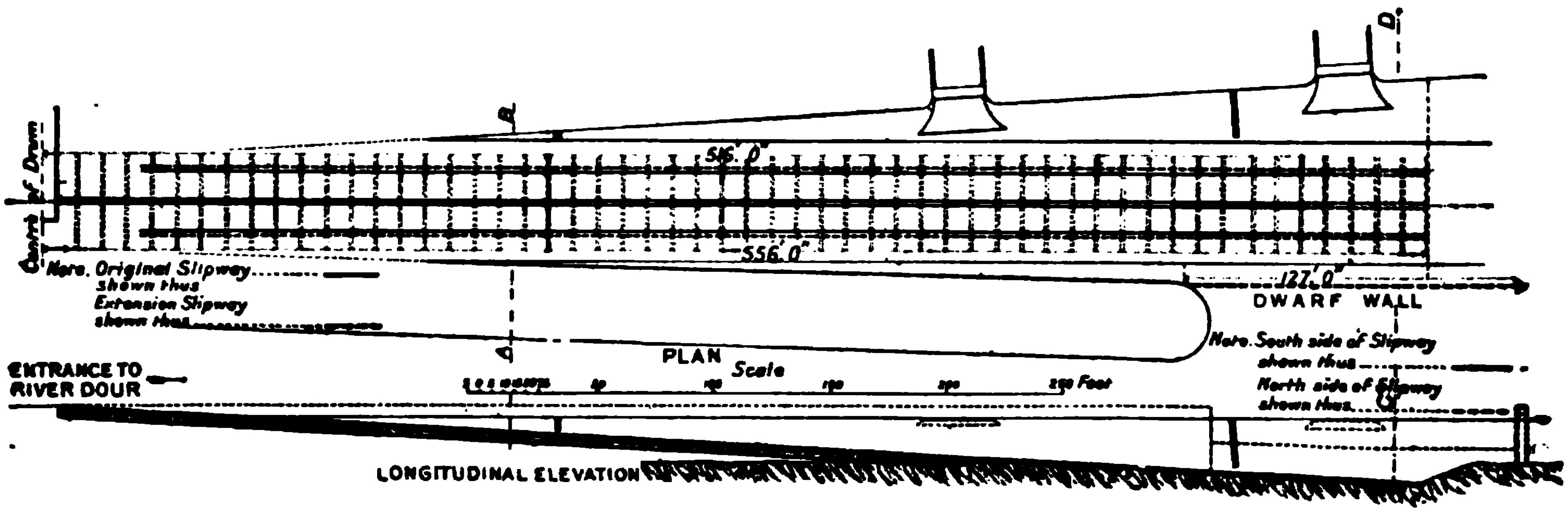
"The upper part of the slipway is in made ground for a length of about 370 feet, and the remaining portion is upon the chalk. In the made ground, a layer of cement concrete, 3 feet 6 inches thick, with an additional foot under the centre pair of rails, is laid at the required inclination. Embedded in this are fir cross-sleepers, 12 inches by 6 inches, 32 feet long and about 11 feet pitch, carrying the longitudinals to which the cast-iron rails are trenailed. In the lower portion of the slip, the cross-sleepers are laid directly upon the chalk, with only enough concrete to bed them evenly, and are held down by six bars, about 2 feet long, driven into the rock. The upper part of the slip, for a length of about 260 feet above low water mark, is paved with Kentish rag, flush with the tops of the longitudinals, and the lower portions with bricks on end."

There are three pairs of cast-iron rails, in 10-foot lengths, and weighing 69 lbs. per lineal yard, the centre pair and rack plate being in one. The single rails are bossed out at the ends and centre, and secured to the longitudinals by six trenails, tbe double rail being secured by trenails on each side of the rack plate.

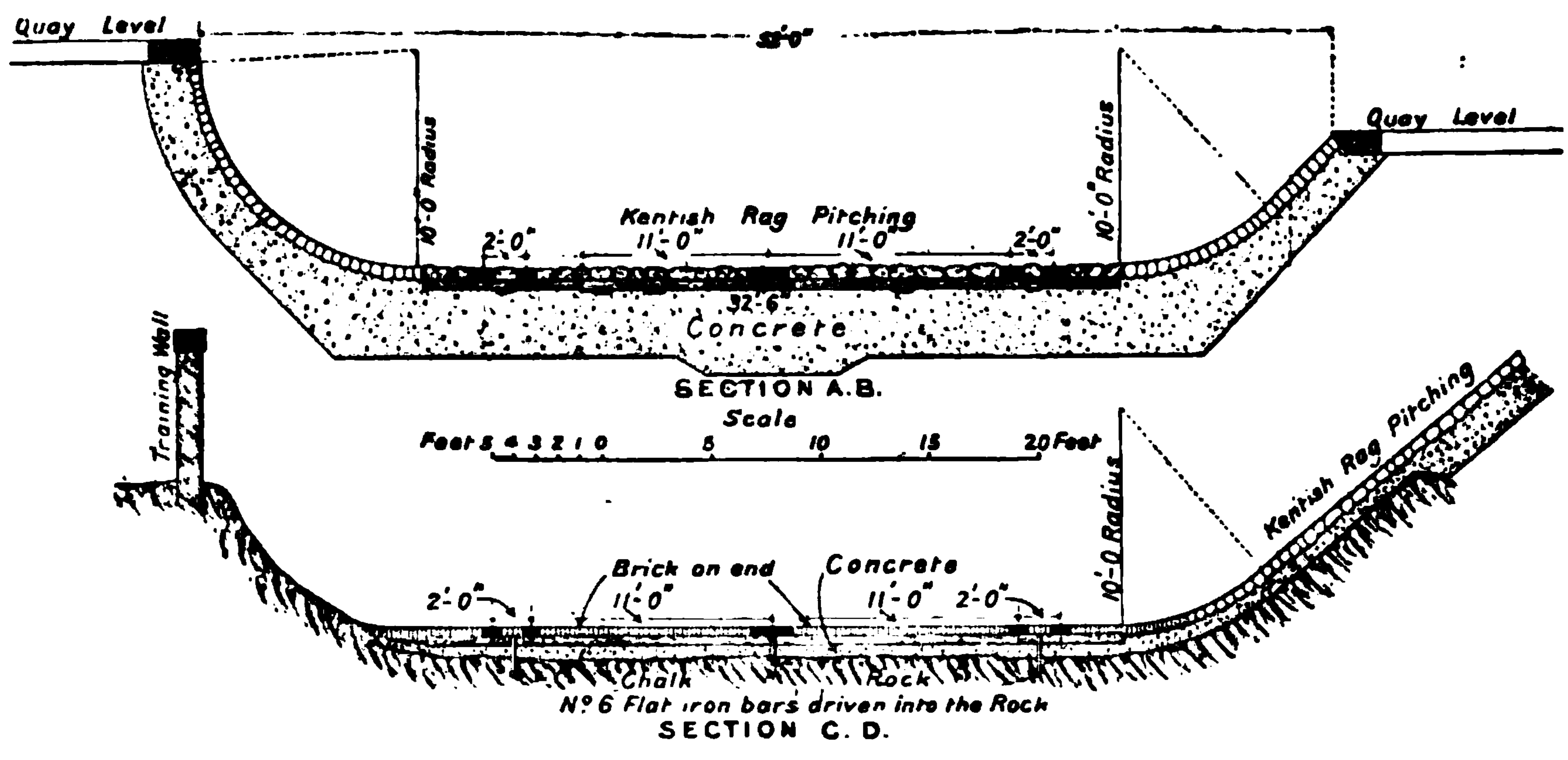
The cradle is in four sections—two of them, forming the main cradle, are together 133 feet long, the remaining two being auxiliary pieces, 25 feet

* Beer on "Ship Slipways," *Min. Proc. Inst. C.E.*, vol. cxviii.

and 15 feet long respectively. The main cradle consists of three lines of longitudinals, supported upon rollers, travelling on six lines of rails, carrying seven pairs of cross pieces or bilge-cods. The centre longitudinal is made up of two 10-inch by 6-inch pitchpine timbers, bolted together with a 5-inch by $1\frac{3}{4}$-inch flat iron bar between them, and 5-inch by $1\frac{1}{4}$-inch bars on each side. The central iron bar runs the whole length of the main and auxiliary cradles, and is increased to $2\frac{1}{2}$ inches thickness above the first section of the main cradle. The side bars run through to within 26 feet of



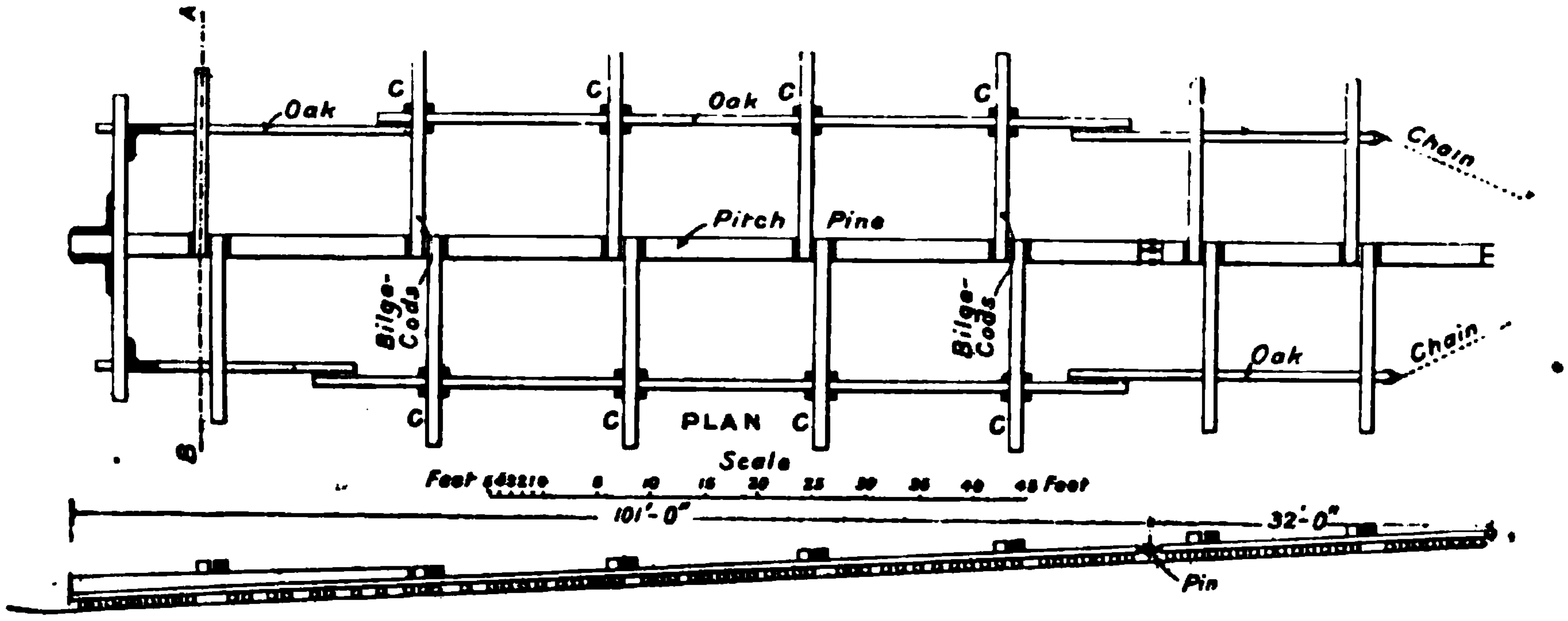
Figs. 484 and 485.—Plan and Elevation of Slipway at Dover.



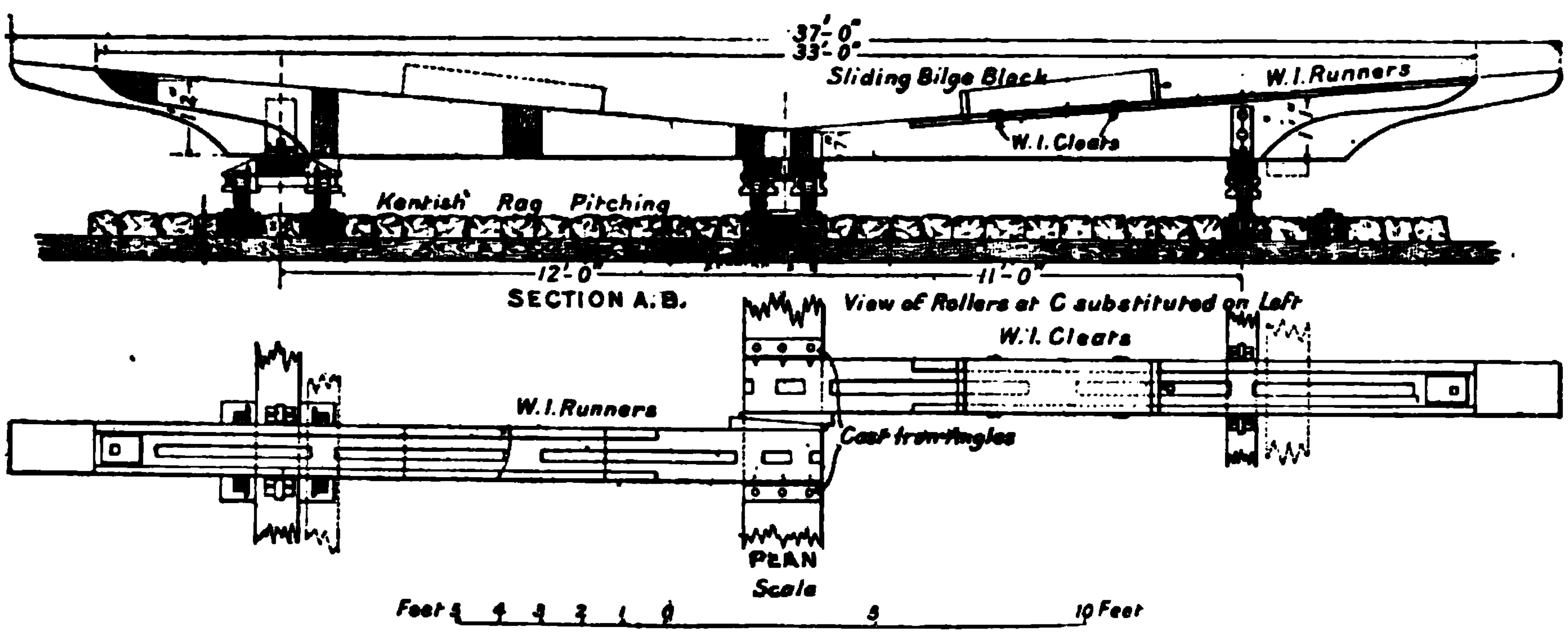
Figs. 486 and 487.—Dover Slipway.

the bottom of the cradle. The centre longitudinal is supported by 83 pairs of rollers, placed directly underneath the main cradle, and the auxiliary cradles, by rollers about 3 feet apart. At the centre portion of the main cradle, the side supports are formed of 12-inch by 5-inch longitudinals, carried by two pairs of rollers under each bilge-cod at the upper and lower ends; the longitudinals are jointed on the inside, so as to bring them directly over the inner rails of the outer pairs, and they are carried by two single rollers on each side, under the bilge-cods.

The cross-pieces, or bilge-cods, are of oak, the four central pairs being 37 feet long and the remaining three, 33 feet long. They are secured to the centre longitudinal by placing the ends of a pair of cods together, and wedging out against two cast-iron knee-pieces with small teeth on their faces, these fit into holes in the cods. The bilge-cods are shaped on the upper face to a slope of about 1 in 14, and for the greater part of their length, strips of iron, 3 inches wide, are let into them and upon these run the sliding bilge-blocks. The longitudinals at the bottom of the cradle are framed into oak cross-pieces, and are stiffened by four cast-iron brackets.



Figs. 488 and 489.—Plan and Elevation of Cradle.



Figs. 490 and 491.—Details of Cradle.

At about every 20 feet in length of the main and auxiliary cradles, a pawl is fixed under the centre beam, working in the rack between the rails. When not in use, it may be lifted up into a horizontal position. The auxiliary cradles have no bilge-cods or blocks. Chains are fixed to the sides of the main cradle and attached to the auxiliary lengths near the centre. They have a sectional area of 1 square inch and are provided with adjusting screws. The rollers are 8 inches in diameter, $3\frac{1}{2}$ inches wide on the face, with a flange $\frac{7}{8}$ inch deep. They are of iron, cast round a $1\frac{5}{8}$-inch Bessemer steel shaft, bossed out and roughened in the centre. The wrought-

iron draw-rods are double and have a minimum sectional area of 16 square inches, a length between centres of 12 feet 6 inches, and a total length of 312 feet 6 inches. There is also a draw-chain, 49 feet long, with the same sectional area.

The Kaiser Graving Dock at Bremerhaven.*

This dock was built by the State of Bremen, between 1896 and 1899, to accommodate the large new ships of the North German Lloyd, to which Company it has been let. It is entered from the "Kaiserhafen," which itself is connected by locks with the estuary of the River Weser. It is illustrated in figs. 492 to 499.

The maximum available nett length of the dock, measured at the level of the keel-blocks, is, in round figures, 741 feet. The dock in this case is closed by a floating caisson, placed outside and bearing against the square quoins of the pierhead of the entrance. In its normal position, however, the caisson is berthed 13 feet further inwards at grooves provided midway in the entrance, and, when in this position, the nett length of the dock is only 728 feet. There is yet a third sill, with corresponding grooves for the caisson, within the dock and enclosing a length of 545 feet.

The side walls of the main entrance have a batter of 1 in 4, and the mean width of the entrance is about 92 feet. The sill of the dock is laid 8 inches below the sill of the entrance lock between the river and the Kaiserhafen, and is 23 feet 6 inches below the local zero. Ordinary high water is 11 feet 9 inches above zero, giving a draught over the sill of 35 feet 3 inches; on extremely rare occasions, however, the water in the wet dock may fall to 6 feet 6 inches above zero, and the available draught then becomes 30 feet.

The width of the dock bottom has been arranged so as to leave a clearance of 6 feet for workmen on each side of the hull of a vessel, 82 feet wide. The central strip upon which the keel-blocks rest has, like these, a fall of 1 in 600, at the side there is a fall of 1 in 450 towards the inlet channels of the pumping station well, which are placed in the western side wall of the dock behind the inner sill. The floor was subsequently raised for a length of 98 feet at its inner end, so that workmen who are engaged in repairing a ship's screws, can start upon their work without waiting for all the water to be pumped out. The height of the keel-blocks is 3 feet 6 inches, and this also represents the depth of the dock below the sill.

The dock is closed by a floating caisson, which only differs from those of ordinary construction in that it carries a 20-ton crane. The dock can be closed in twenty minutes.

The keel-blocks are entirely of timber, spaced at 4 feet 6 inches centres, and have a base area of 6 feet by 20 inches. The upper portion consists of oak logs, bolted together, and the lower portion of pitchpine timbers,

* Rudloff on "Docks," *Int. Nav. Cong.*, Dusseldorf, 1902.

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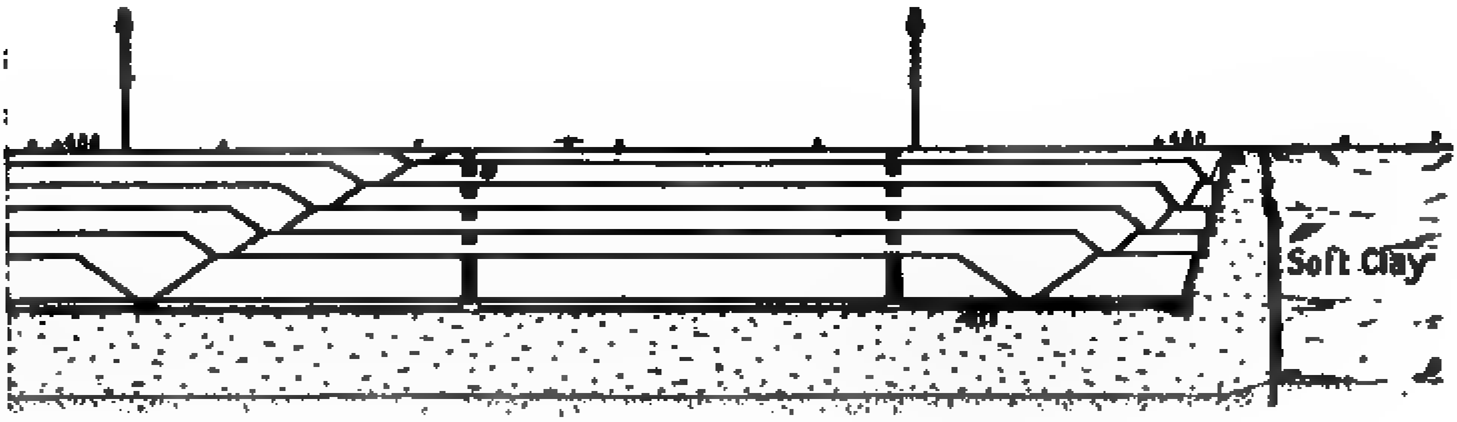
tion A—B



er Graving Dock at Bremerhaven.

secured in the same way. The bilge-blocks, for supporting the bottom of a ship on both sides of the keel, consist of strong pitchpine timbers, arranged scaffold fashion, resting upon cradles, which are drawn under the ship and adjusted by wire ropes, passing through rollers. The cradles move on special slides bordered with iron, and are spaced 27 feet apart.

The graving dock is constructed parallel to and adjoining a large sea-lock, which, generally speaking, was built under the same conditions as the dock. The experience, gained during the former undertaking, proved very useful in carrying out the second without any serious interruption. The work proceeded in the following manner:—

Preliminary operations consisted in digging out trenches to a depth of 3 feet 3 inches below zero or about 14 feet 9 inches below ground level, by means of a land dredger, and the excavated material (soft clay) was carried off in tipping waggons and utilised for raising the ground all over the harbour site. In the trenches thus formed, 12-inch sheet piling was driven, averaging 55 feet in depth, to enclose the dock foundation proper. A second row of piles was driven at the same time to serve as anchorage to the sheet piling. When this had been done, water was admitted into the trenches from the harbour, and excavation was continued down to the bottom of the foundation by means of a floating bucket-dredger. But as the latter was incapable of working at a depth of 58½ feet, and as grabs did not work satisfactorily, the trenches had to be closed again by dams and pumped out for the concluding portion of this work.

After the trenches had been carried down to the required depth, water was admitted to them a second time, and the bottom layer of concrete, composed of 1 part of lime, 1 of trass, and 1 of sand to 4 parts coarse river gravel from the Weser, deposited in skips. The whole of the foundations were completed in 15 weeks at the rate of 800 cubic yards a day, the maximum output being 930 cubic yards in twenty hours. The average thickness of the foundation was 19 feet 6 inches.

The layer of concrete was left undisturbed for a period of three months, after which it was pumped dry and levelled to an even surface, being further strengthened with strong iron bands, to prevent its breaking up. The building of the walls was then proceeded with. They were mainly constructed in concrete, with a hard clinker facing and granite copings, quoins, and bedstones. No leakage occurred through the concrete foundation, but a strong flow through a gap in the sheet piling was conducted into the pump well without giving further difficulty.

The pumping plant consists of two 49-inch centrifugal pumps for emptying the dock, and two 10-inch centrifugal pumps for dealing with the leakage water. The former set are driven by special, direct-coupled, triple-expansion engines. Each pump can lift on an average 150 cubic feet a second, and as the dock holds 2,700,000 cubic feet, it can be emptied in 2½ hours. The drainage pumps are driven by 30 H.P. compound engines. Only one drainage pump is needed, as a rule, and that intermittently.

There are two culverts for filling the dock, one in each side wall, with a sectional area of 87 square feet, closed with vertical paddles of the ordinary type, working in granite grooves. Each paddle consists of a built-up mild steel frame, covered with tongued and grooved oak planks and provided with greenheart rubbing fillets.

Canada Graving Dock at Liverpool.

The extreme length of the dock (figs. 500 and 501) from point of sill to head of dock is 925 feet 6 inches. It has an entrance width of 94 feet and a depth of water of 32 feet at high water of ordinary spring tides. The height of the pierheads is 41 feet above sill level. In the interior of the dock the bottom width is 94 feet, whence the side walls recede in a series of thirteen offsets, or altar courses, of irregular height, to a width of 124 feet 2 inches at coping level. The coping level is 35 feet 8 inches and 36 feet 1 inch above sill level on the north and south sides of the dock respectively. Communication with the bottom is made by means of six sets of stone steps and slides, three at each side, and there are also two stairways at the head of the dock.

The floor of the dock has a fall of 9 inches from the centre to the sides, where drainage channels communicate through 18-inch drain pipes with the two central culverts. These are parallel to each other, and to the longitudinal axis of the dock, commencing with a section, 4 feet by 3 feet, of which the sides are vertical and the roof and floor curved, passing through the circular form with 5 feet 6 inches diameter, and finally assuming an egg-shape, 8 feet deep. A rectangular pit, 12 feet by 35 feet open save for an iron grating, receives the water from the dock at its north-west corner, whence it is transmitted by two large rectangular culverts, each 10 feet 9 inches by 9 feet, to the pumping well at a level of 18 feet below the sill.

The pumping plant consists of three 51-inch centrifugal pumps, each driven by a condensing engine of 700 H.P., with two high-pressure cylinders, 25 inches diameter and 2 feet stroke. Steam at 110 lbs. pressure is supplied from six sets of Babcock and Wilcox patent water-tube boilers, having 3,116 square feet of heating surface and 59½ square feet of grate area to each boiler. The pumps are capable of lifting 1,000 tons per minute, and of emptying the dock, whose capacity is 3,226,648 cubic feet, in an hour and a-half. There is also a small 14-inch drainage pump for dealing with leakage, which is very slight.

The bulk of the walls and floor are of concrete, composed of 1 part of Portland cement to 6 and 8 of Harrington gravel, faced with 2 to 1 concrete, and having granite coping, quoins, sills, steps, and slides. The keel-blocks are of cast-iron wedges, surmounted by a birch cap 12 inches thick. The top of the blocks is 4 feet above the floor level. The entrance is closed by greenheart gates, and the clough paddles are also of greenheart. Behind the gate heel-posts are two culverts for filling the dock.

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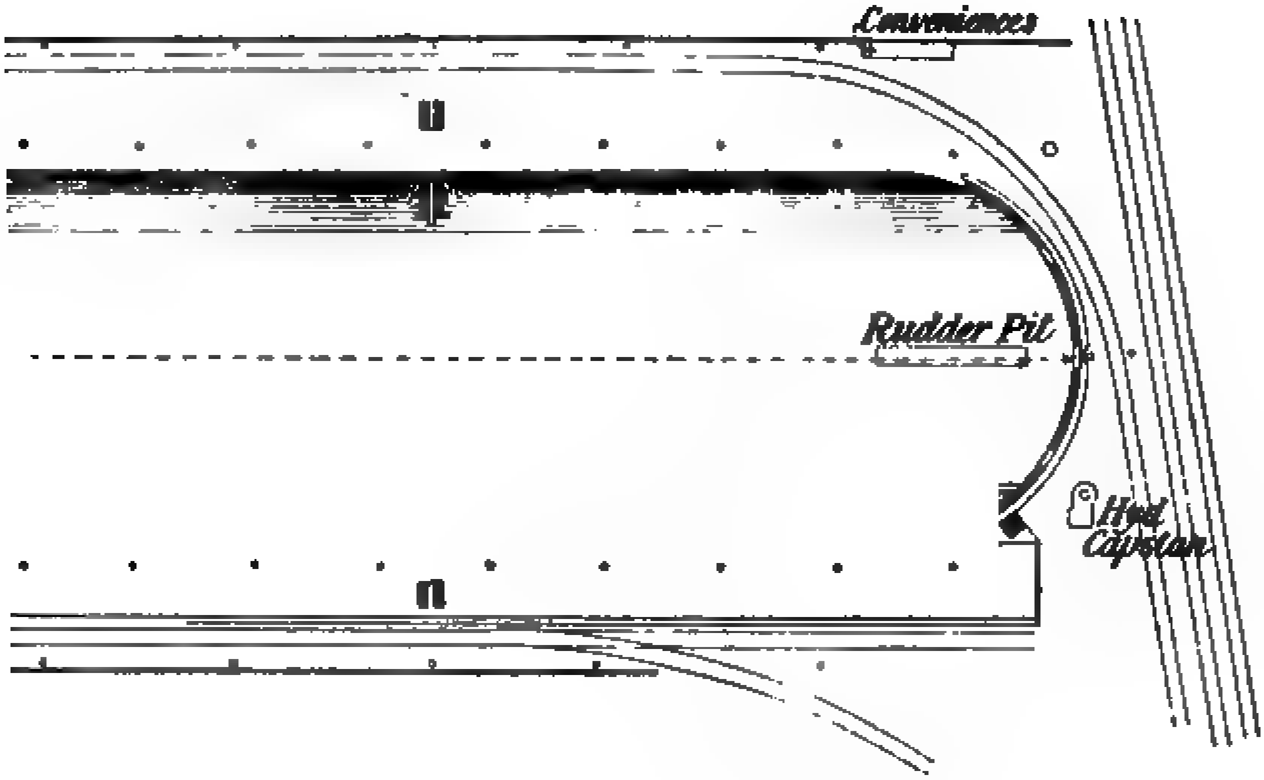

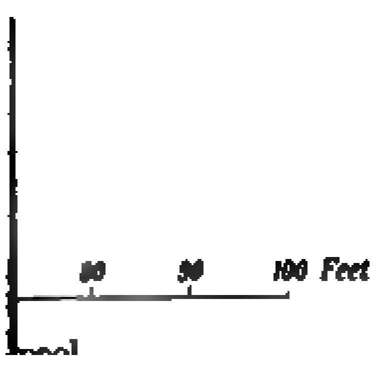

The equipment consists of the usual mooring posts, shores, capstans, and mushrooms, together with a 40-ton hydraulic crane erected by Messrs. Geo. Russell & Co., of Motherwell.

No difficulties of any serious importance were encountered during the construction of the dock. The site lay to the eastwards of the Canada Dock, a short distance behind the east wall of which, the main bulk of the excavation, principally clay, interspersed with beds of sand, was done under normal conditions with a steam navvy and grabs. When this operation had been completed as far as possible, and the walls, floor, and sill put in, a dam of 12-inch sheeting piles was driven across the front of the entrance and was shored in the first instance to the masonry of the east wall. The water having been pumped out between the dam and the wall, the pierheads were put in. Then the old east wall was gradually demolished, the bearings of the shores being transferred to the pierheads and the sill. Finally, water was let into the dock, the dam removed, and then some little dredging at the entrance completed the undertaking.

No. 3 Graving Dock, Glasgow.*

This dock (fig. 502) opened in 1898, has the following general dimensions:—

	Ft.	Ins.
Length of floor, from inside of caisson at outer entrance to head of dock,	880	0
Width at bottom,	81	8
Width at top,	115	0
Width of outer entrance at bottom and top,	83	0
Width of inner entrance at bottom and top,	83	0
Depth on centre of sill of outer entrance at H.W.O.S.T.,	26	6
Depth on centre of sill of inner entrance at H.W.O.S.T.,	27	0
Level of floor of dock below H.W.O.S.T.,	28	6
(except at gate chamber, where it is 6 inches deeper.)		

The dock is divided, by a pair of steel gates, into two lengths of 460 and 420 feet.

The strata underlying the site of the dock consisted mainly of fine sand and gravel with occasional pockets of clay. The structure rests upon moving sand, and the wing walls and apron are founded on triple-concrete cylinders, in the manner described in Chap. v. Two of the cylinders of the apron remained unfilled till the dock was nearly completed, being used as sumps for pumping purposes. Into these wells 9-inch pipes, bedded in clean-riddled gravel, were led, in order to drain the dock area which was excavated to low-water level with side slopes, but below that level excavation was carried on within sheet piling, 44 feet long by 12 inches thick, driven along the sides and round the upper end of the dock.

* Alston on "Glasgow No. 3 Graving Dock," *Int. Eng. Conf.*, Glasgow, 1901; also *The Engineer*, May 20, 1898.

The floor foundation consists of a bed of concrete, 14 inches thick at the centre and 4 feet 6 inches thick at the sides. On this was laid the brick invert, 5 feet 10 inches thick, with a radius of 177 feet, surmounted by a bed of concrete 6 feet 6 inches thick at the centre, diminishing to 12 inches at each side, with a cross-sectional camber of 6 inches on the upper surface. The surface finishing consists of a 6-inch granite causeway, with the exception of a length of 103 feet at the head of the dock, which was paved with granite blocks.

The side walls of the outer and inner entrances and the head of the dock are of brick and concrete. The walls of the outer entrance were faced with granite and those of the inner entrance and the head of the dock, with moulded granolithic-faced ashlar, all coped with granite.

Fig. 502.—No. 3 Graving Dock, Glasgow.

The side walls of the dock proper are of concrete, put in between movable frames, roughly stepped to receive the granolithic altar courses, fourteen in number, ranging in dimensions from 45 by 20 inches to 18 by 14 inches. The altar courses, with the exception of the bottom course, were made in moulds on a platform and then built in position like ashlar; the bottom course was made *in situ*. The side walls are 4 feet 6 inches lower than the pierheads of entrance. Four double stairs of granite, with granite timber slides, are arranged in each of the two divisions of the dock.

Near the outer entrance there is a rudder well, 10 feet long by 7 feet wide by 8 feet deep.

The apron for the sill of gates of inner entrance is 6 inches below the floor of dock at the centre. The sill is 18 inches above the floor at its centre. The meeting faces and inner side of the sill are of granite. The upper surfaces of sill and apron are paved with granite, and in the apron are bedded

radiated granite stones to carry the cast steel roller paths, 9 inches wide, for the gates.

The outer division is filled by two culverts, 7 feet 4 inches high by 4 feet wide, one of them passing round the caisson chamber. The inner division is filled from the outer division by two similar culverts. A loop culvert, 6 feet by 3 feet 6 inches, is also provided, leading from the main discharge culvert into a sump to increase the rapidity of filling.

For emptying the dock, a sump or well, 61 feet long by 12 feet broad by 11 feet deep, is constructed under the engine-house, and the water is discharged therefrom into Princes Dock through a main discharge culvert, 11 feet 6 inches high by 8 feet wide. The pumping installation consists of two 60-inch centrifugal pumps, each driven by a pair of vertical direct-acting engines, with cylinders 28 inches diameter and 24 inches stroke, the steam pressure being 110 lbs. per square inch. An auxiliary 15-inch pump deals with leakage water. Steam is supplied from four boilers of the return tubular marine type, with assisted draught, each 12 feet 6 inches diameter and 10 feet long, giving a working pressure of 130 lbs. per square inch.

The capacity of the dock when no vessels are in, is about 2,202,000 cubic feet or 13,762,500 gallons at high water. This quantity can be discharged in 1 hour 40 minutes.

The equipment consists of a 25-ton steam travelling crane on the north side, with sweep to centre of dock; five 5-ton direct-acting hydraulic capstans and 12 hand capstans, together with 31 mooring posts. The sets of portable keel-blocks, 620 in number, are laid down in three lines. Their general sizes are 5 feet long, 16 inches broad, and 30 inches high.

The Portland cement concrete was of the following proportions:—For the foundation cylinders, 5 to 1; for floor and sides of dock, 6 to 1; for pockets in brickwork of entrance and end walls, 9 to 1; for moulded granolithic-faced ashlar, 6 to 1; and the granolithic facing, 3 to 1.

Commercial Graving Dock at Barry.*

The general dimensions of this dock, completed in 1893, are as follows (*vide* figs 503 and 504):—

	Ft.	Ins.
Extreme length,	747	6
Outer compartment,	384	6
Inner compartment,	363	0
Width of outer entrance at coping level,	60	3
Width of inner entrance at coping level,	59	4½
Depth of water on outer sill at H.W.O.S.T.,	26	8½
Depth of water on inner sill at H.W.O.S.T.,	27	2½
Width of dock chambers at coping,	113	6
Width of dock chambers at floor,	100	0

The earthwork consisted chiefly of red marl. Magnesian limestone was also found, and was used in the dry rubble drains behind the walls and for

* Robinson on "The Barry Graving Docks," *Min. Proc. Inst. C.E.*, vol. cxvi.

backing. Numerous joints and fissures were exposed by the excavations in the hard marl and limestone, and from some of these salt water issued. Under the foundations of the west wall of the inner dock a cavern, 27 by 23 feet and 14 feet deep, was discovered in the marl, through a hole in which, sea water burst forth and continued to flow at each tide, but ceased at low water of spring tides. A brick wall in cement mortar was built round the hole at the top of the cavern to keep in the water. Rubble stones were then deposited in the cavern, and a Portland cement concrete floor, 6 feet thick, was laid on them, and on this the wall is founded.

The floor is of Portland cement concrete, 6 to 1, and 2 feet in thickness, with stone drains. Across the floor, in and under the concrete, 4-inch land drains are laid, 20 feet apart, to convey any rising water to the open drains. The walls are built of mountain limestone, weighing 169 lbs. per cubic foot, from the Alps quarry, about 5 miles away. The roughly-dressed face-stones are squared but not laid in courses, and have close beds and joints for 6 inches, lipped with cement for 3 inches inwards at the time of building. The remaining portions of the walls are built in blue lias lime mortar. Headers, not less than 3½ feet long, pass through from front to back and overlap each other. The altars, 2 feet by 9 inches, and the coping are of granite. The depth of the dock from coping level to floor is 32 feet 6 inches. The walls have not been designed to resist water pressure from the back, and cast-iron pipes are inserted in them to allow the water to escape. Any reflex action is prevented by brass clack-valves. There are wrought-iron ladders connecting the altar courses and flights of wooden steps in the corners for access to the floor.

The walls of the entrance have a batter of 1 in 8. The sill stones and caisson quoins are of granite, fine-axed on the meeting face. The bearing blocks for the bottom of the caisson are limestone, 2 feet by 15 inches, standing 1½ inches above the concrete floor, 6 feet apart centre to centre, and level throughout. Hydraulic pipes and electric light mains are carried in a recess, 8 feet wide and 18 inches deep, in the walls and across the invert. The last-named is built in brickwork with cement mortar, faced with two courses of Staffordshire blue bricks.

Beneath the engine and boiler house (fig. 505) are the suction and discharge chambers, 10 feet wide and 11 feet high, with floors 3 feet 6 inches lower than that of the graving dock. Two culverts, 6 feet 6 inches in width and height, conduct the water to the suction chamber and a similar sized barrel-culvert conveys the water from the suction chamber to the sea. There are three suction pipes in the pumping chamber, 33 inches diameter, and three similar discharge pipes in the discharge chamber. In addition, there are two suction pipes and one discharge pipe, 12 inches diameter, connected with the two drainage pumps. The pumps consist of three 33-inch horizontal, high-pressure condensing, centrifugal pumping engines and two 12-inch drainage, centrifugal pumping engines, supplied with steam from three Lancashire boilers and one auxiliary Cornish boiler. The main

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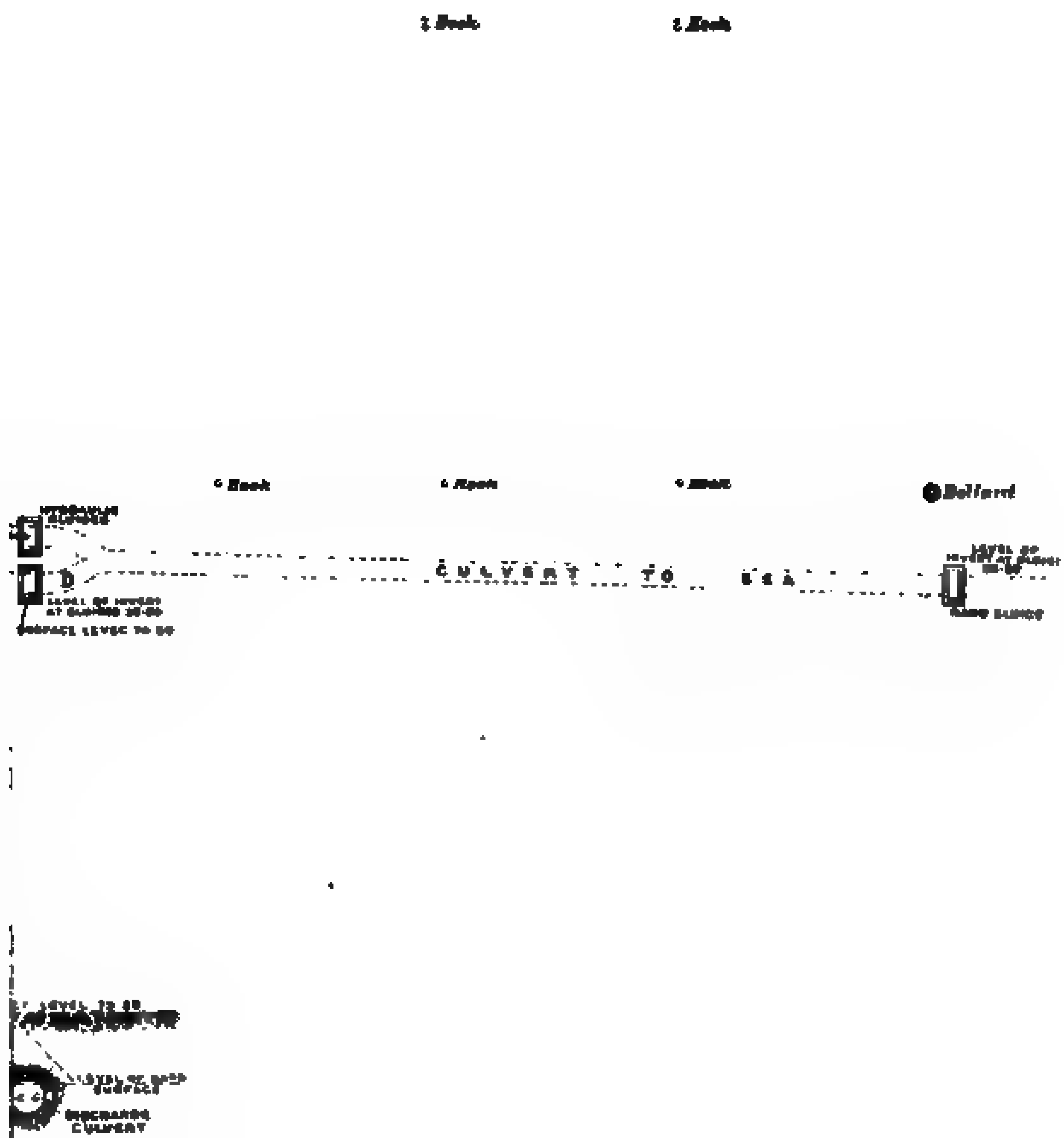

pumping engines have discs, 5½ feet in diameter, and 18-inch cylinders of 16 inches stroke, and they are fitted with variable expansion gear and steam stop-valves. Each pump can make 160 revolutions a minute, discharging 17,000 gallons of water or about 1,000,000 per hour. The dock has been completely emptied in three hours against a head increasing to 22 feet, but both divisions can also be emptied in an hour and a half, by letting the water flow into the sea and in forty minutes when the tide permits.

ENGINE HOUSE
CROSS SECTION

Fig. 505.—Pumping Station, Barry Graving Dock.

The equipment consists of three hydraulic capstans, six bollards, and a number of snatch heads and hooks. The last-named are for giving a slight list to vessels after they have settled on the keel-blocks. The blocks are of cast iron with elm caps, 4 feet long, 3 feet high, and 12 inches wide. They are spaced at 4 feet 6 inches centres and are in two parallel lines, each division of the dock being able to accommodate two vessels side by side. Vessels are supported by timber props from the altars of one side only. They are so arranged that those whose repairs are first completed can leave the dock with the least possible interference to the others. The dock is lighted by electricity.

The entrance and passage are closed by a pair of interchangeable caissons, 17 feet wide and 34 feet 6 inches deep, the top decks of which are planked. A line of railway runs over the dock for locomotives and waggons.

Tilbury Graving Docks, London.

There are two graving docks lying parallel to one another at the entrance to the Tilbury Docks, London * (fig. 506). They are also capable of acting as entrance locks, and for this purpose they are provided with caissons at both ends. In addition to this, there are three central positions in each dock fitted for the reception of a caisson. The result of this arrangement is that, apart from the use of each graving dock in its entirety of 875 feet, there are virtually four graving docks, each complete in itself, two being entered from the tidal basin and two from the main dock. And, by means of a variation in the position of the central caisson, each pair can be adjusted to any of the following lengths in the clear—viz., 450 and 400 feet, 350 and 500 feet, and 300 and 550 feet.

The large graving docks have a width of 70 feet across the bottom and a depth of 35 feet below Trinity high-water mark on the sills. The width of the small graving docks is 60 feet, and the depth on sills 30 feet (figs. 507 to 509).

The walls of the large graving docks have a thickness of 16 feet 3 inches at floor level and of 5 feet at the coping. The backs of the walls are vertical, except where it was necessary to increase the width for culverts, and the internal faces of the walls have a batter of 1 in 20 for a height of 22 feet 6 inches. The upper part of the walls is stepped to form six altars. The thickness of the invert varies with the depth to which it was required to excavate to reach the gravel foundation, but the normal thickness is 15 feet. The walls of the small graving docks are 13 feet 6 inches thick at the base, and their other dimensions are, in general, smaller than those of the walls of the large graving docks in the same proportion. The invert is, however, relatively thicker, on account of the necessity of excavating the foundations to the same depth in both cases. A portion of the invert of the small docks is, owing to a dip towards an old channel, carried upon short whole-timber bearing piles, spaced at 4 feet centres, in each direction. The invert is entirely of 9 to 1 concrete, with a stop-water course, 3 inches thick, of fine 3 to 1 concrete. Upnor clay-puddle, 1 foot 6 inches thick, is carried down the backs of both graving dock walls from Trinity high-water level to below the stop-water course. The floors are of pitchpine planking, 4 inches thick, spiked down to pitchpine sleepers, 14 inches square, which are bedded in concrete. Teak keel-blocks are laid along the whole length of the docks, and are fixed down to the floors by dogs. The altars are paved with 6-inch hard York stone, the copings being of teak, 12 inches square, furnished with eye-bolts and secured to teak cross-timbers, 3 feet 6 inches long, bedded in brickwork at the tops of the walls.

The docks are provided with five hydraulic capstans of 2½-ton and 5-ton power and cast-iron bollards, the latter having perforated caps and being

* Scott on "The Construction of Tilbury Docks," *Min. Proc. Inst. C.E.*, vol. cxx.

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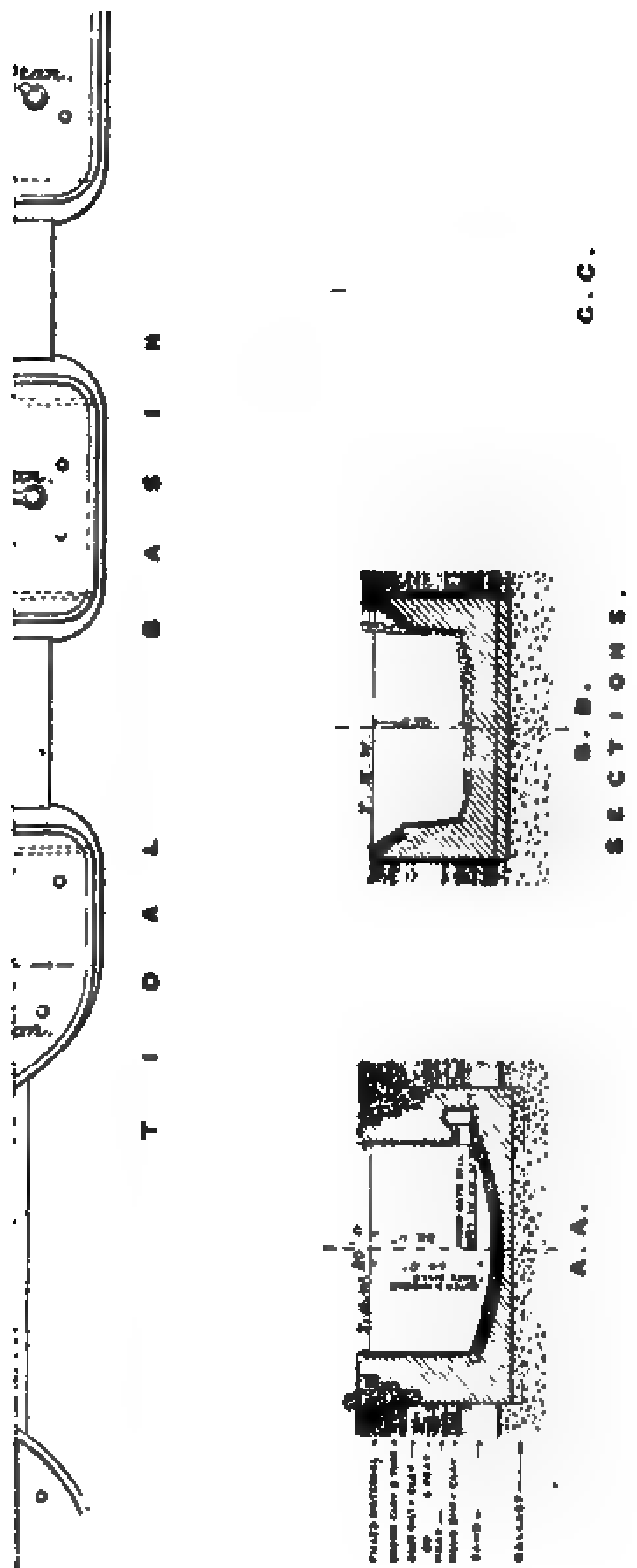

DIMENSIONS.

Lock Length between gates	700 feet.
" Width	80 "
" Depth on sill below T.H.W.	44 "
Large Graving Docks Length	875 "
" Width at bottom	70 "
" Depth on sill below T.H.W.	35 "
Small Graving Docks Length	875 "
" Width at bottom	60 "
" Depth on sill below T.H.W.	30 "

NOTE

Both Graving-Docks are divided by central caissons, forming Docks of 450'.0" & 400'.0", 500'.0" & 350'.0", or 550'.0" & 300'.0".

connected with the various culverts, so as to serve as air-shafts. Each of the four sections of the graving docks has a distinct set of culverts for running out and filling in the water, and the pumping arrangements allow of pumping the water out of any one section into any other section, or, through the discharge pit in the rear of the engine-house, into either the main dock or the tidal basin.

The machinery at the pumping station consists of four centrifugal pumps, two with fans 5 feet in diameter, and two with fans 4 feet 6 inches in diameter. These are driven by four sets of engines of inverted, direct-acting, high-pressure type, two with cylinders 22 inches diameter and 16½ inches stroke, and two with cylinders 17¾ inches diameter and 16½ inches stroke, for the large and small pumps respectively. The pumps are, together, capable of discharging 650 tons of water per minute into the discharge pit, and, therefore, of pumping out the large pair of docks in about one hour. Two distinct sets of double-acting plunger-and-bucket pumps for drainage, each capable of raising 1,000 gallons per minute into the main dock, are provided. The engines driving them, through gearing, are of the horizontal type. Steam is raised from five boilers, one being spare, of modified marine tubular type, 7 feet 6 inches diameter and 20 feet long, with two flues, 3 feet diameter. The flues lead to wrought-iron chimneys, one for each boiler, and thus an ordinary shaft is dispensed with. Forced draught is driven through the stokeholds by five fans, each with a small independent engine. A cast-iron tank of 250,000 gallons' capacity, into which water from the drainage culverts is pumped by an auxiliary engine, covers the boiler-house.

Floating Dock at Bermuda.*

The new dock at Bermuda (figs. 510 and 511) launched in 1902, to replace the former dock of 1868, is from designs by Messrs. Clark and Standfield. It is 545 feet long, with a clear width of 100 feet between the rubbing fenders. The side walls are 13 feet in width, which gives a total width to the structure of about 126 feet. The lifting power, up to the pontoon deck level, is 15,500 tons, but, by using the shallow pound, this can be increased to 17,500 tons. The weight of the hull is 6,500 tons. The sides are high enough to enable a vessel of 32 feet draught to be berthed on the keel-blocks, the latter being 3 feet 6 inches high. The whole structure consists mainly of five parts—three floor pontoons and two side walls. The pontoons supply the chief part of the lifting power, and though the side walls may be used to some extent for the same purpose, their primary object is to give the structure stability and to afford control over the dock in sinking it to take the ship on board. The end pontoons are each 120 feet long and are bevelled in such a way as to facilitate towing. The centre pontoon is 300 feet long. The sides of the pontoons

* Vide *Engineering*, February 14, 1902.

are built up above the deck level to a triangular profile so as to form three altar courses.

The side walls are each 435 feet long and 53 feet 3 inches high, and, in order to give sufficient space for the boilers, they are sponsoned out, forming an upper chamber, 12 feet 6 inches wide. There are four large openings in the walls for the purpose of affording light and ventilation under the bottom of a docked vessel.

The three pontoons are subdivided into 40 pumping compartments, and of these 32 are watertight. There are also eight watertight compartments in each side wall. All these divisions are provided with a separate pipe and valve, the pipes leading directly into the two main side drains. The drains are continuous throughout the length of the walls, and as the four 18-inch centrifugal pumps are seated directly on them, any one pump can empty all the compartments of its half of the dock. There is a central bulkhead, dividing the dock into two halves, but this is not quite watertight, small leakage holes being purposely left. If, therefore, the whole of the pumping machinery on one side were to break down, the other half could still empty the dock, though at a somewhat slow rate. The pumps are driven each by a separate compound condensing engine directly attached. A return-tube boiler supplies each pair of pumping engines with steam; but the connections are so made that the supply of steam from any boiler is interchangeable.

The working of the whole dock is done from two central positions on the top of the dock towers, where the valve wheels and connections are placed, with indicators to show the condition of the valves, whether open or shut. There are six capstans for warping ships into position, with the usual bollards, fairleads, &c. Lighting at night is done by electricity with 12 arc lamps, beside smaller services. Two 5-ton travelling jib cranes are worked by the same motive power from separate generating plants placed in the dock towers, the leads being mutually interchangeable.

The underside of the dock is protected by a series of greenheart keels, as it is possible the dock may ground at low water, and the bottom of the harbour at Bermuda is of coral. The top decks are planked with teak.

Figs. 512 to 515 are views of the dock in its various positions.

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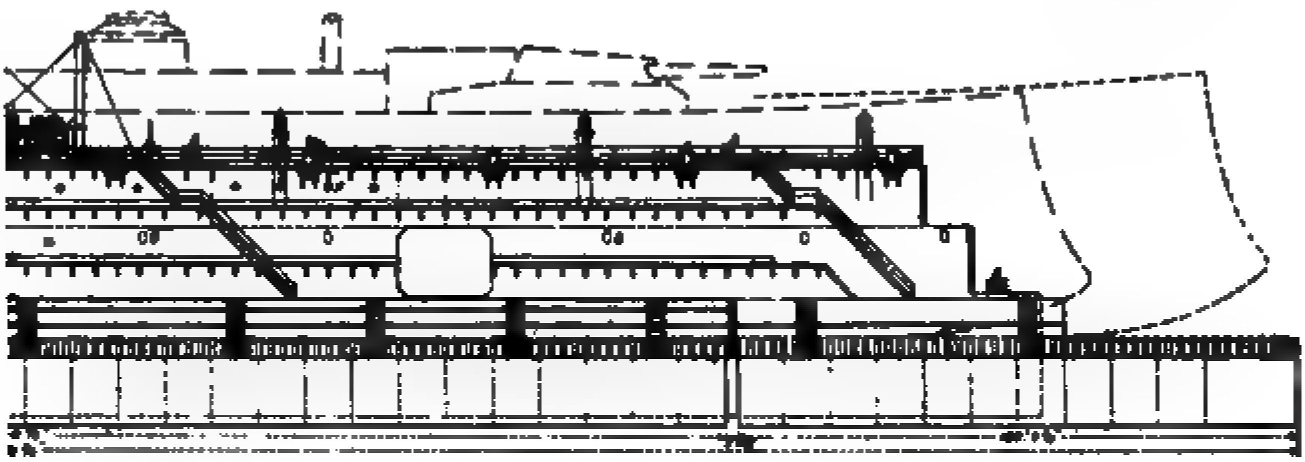
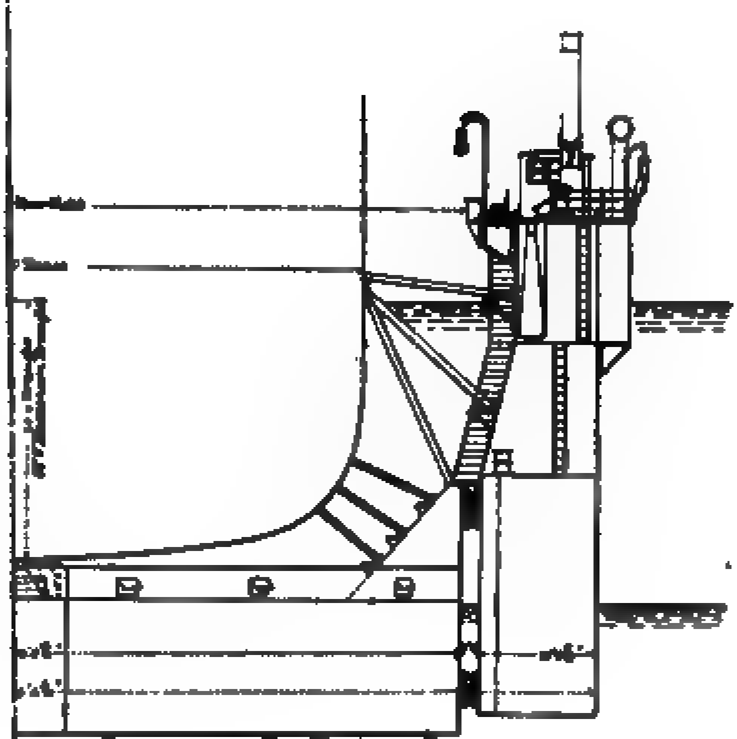
XXV.





muda Floating Dock.

Fig. 512.—Dock Heeled to remove Starboard Connection.

Fig. 513.—Centre Pontoon Floating Free. Rest of Dock being Sunk.

Fig. 514.—Dock Sunk, bringing Centre Pontoon Connection into Upper Position.

Fig. 515.—Centre Pontoon connected to Dock in Upper Position and lifted Clear of the Water.

CHAPTER XII.

WORKING EQUIPMENT OF DOCKS.

SOURCES OF POWER—COMPRESSED AIR—STEAM—WATER UNDER PRESSURE—ELECTRICITY—COMPARATIVE EXPENDITURE OF ENERGY—CRANE TESTS—COST OF POWER—HYDRAULIC MACHINERY—SYSTEMS OF ELECTRICAL DISTRIBUTION—APPLICATIONS TO DOCK EQUIPMENT—GATE MACHINERY—POWER OF GATE MACHINES—SLUICING MACHINERY—CAPSTANS—QUAY AND FLOATING CRANES—JIGGERS AND TRANSPORTERS—COAL TIPS AND LIFTS—GRAIN ELEVATORS—SLIPWAY HAULAGE—PUMPING INSTALLATIONS—PETROLEUM STORAGE—GENERAL EQUIPMENT—LIST OF APPLIANCES IN USE AT HAMBURG, HAVRE, AND LIVERPOOL.

THE subject of dock equipment is scarcely less extensive, and certainly no less important, than that of dock construction, strictly so-called. Indeed, the two departments are so intimately associated in aim and development, that they cannot well be separated, and a technical work which pretends to any completeness of treatment, must inevitably include not only an outline of the nature and functions of the various appliances included in the working equipment of a dock system, but also some description, however succinct, of their essential parts. Any elaborate investigation appertains, of course, more appropriately to the domain of the mechanical, and, often in these later days, the electrical specialist; but, without some general knowledge of the subject, a dock engineer would be manifestly imperfectly fitted to discharge the duties and responsibilities of his position.

Before proceeding to a categorical analysis of the machinery in question, it will be well to devote a few preliminary remarks to the broad question of sources of power—their availability, utility, and economy, for the respective purposes held in view.

Power.—The power employed for actuating dock machinery is derived from four sources:—

1. Compressed air.
2. Steam.
3. Water under pressure.
4. Electricity.

Strictly speaking, all but the second of these agencies are mere transmitters of power already in existence. As a matter of fact, all present forms of power have their practical origin in the steam engine,* by which

* The waterfall and windmill are ignored as too limited in application and as unlikely to be resorted to in connection with dockwork. Internal combustion engines, such as the gas engine, despite their great potentialities and rapidly increasing use, have not yet acquired sufficient importance as prime movers to bring them into active competition with the steam engine. The day, however, is not far distant when they will gain a very prominent position in this respect.

electricity is generated, and air and water are pumped under pressure. This distinction, however, is not of sufficient moment to call for more than a passing remark, and need not invalidate the tabular arrangement adopted above, of which it will be convenient to take each item in detail, *seriatim*.

Compressed Air.—Air, like steam, is an elastic fluid, and, consequently, in its capacity as a transmissive medium, has the advantage of accommodating its volume to the resistance of the load—in other words, the work done is commensurate with the power employed. But this alteration of volume entails corresponding disabilities. Compressed air never effectively reproduces all the work which is done upon it; partly, because it is not capable of expansion to the same extent as its previous compression and, also, because some of the energy imparted to it is dissipated in the form of heat. Then, again, leakages are rapid and difficult to detect, so that in long lines of communication there is inevitably much loss.

Apart, however, from these drawbacks to its use on a large scale, compressed air has many advantages to offer for the working of small portable appliances, such as those employed in connection with ship repairs in graving docks, and the fact that sufficient power for the purpose can generally be obtained from a small air-pump renders it desirable, in the absence of more important installations, to equip such docks, especially if in isolated situations, with a pump, pipe lines, and branch couplings, so that the pressure may be transmitted readily to any desired point. This, however, apparently marks the limit of utility of compressed air in connection with dockwork.

Steam.—The most useful characteristic of steam power is the convenience with which it can be adapted to detached locomotive machinery. It necessitates no central generating station, although such can be employed in cases where the circumstances render it permissible. The general practice is for each machine to be entirely independent and self-supplied. In this way the loss of energy arising from long lines of communication and multiple connections is entirely obviated. Steam has the further advantage of supplying each machine with its own means of mobility, whereas in the case of other systems conforming to the exigencies of dockwork, * transportive power has generally to be obtained from extraneous sources. On the other hand, for intermittent operations, unless carried out in connection with a central station, steam power is not always readily available, nor indeed without due preparation. A boiler has to be heated, and some delay is inevitable before the requisite pressure is obtained; furthermore, there is considerable waste of heat in the cooling down of the boiler after the allotted duty has been performed.

A central generating station certainly does away with these defects, but the loss of heat from the steam supply due to its transmission through pipes to outlying positions is excessive, so much so that in no case will any

* Trolley wires and underground cables are considered inapplicable to these special conditions.

advantage be derived if the point of application be situated more than 350 to 400 yards from the point of generation. Even at less distances, a machine will be but indifferently served.

These considerations all point to the conclusion that steam is an admirable motive agency for locomotive cranes and other appliances in which, in addition to local action, movement through an extensive range of position is essential, but that in order to be economical such machines must be at work continuously for long periods. It has advantages, also, for small detached installations, where the cost of a centralised generating plant, with extensive ramifications, would be out of proportion to the duty required. In all other cases, a system of hydraulic or electrical energy will be found preferable.

Water under Pressure.—In contradistinction to the previous elements, water is an incompressible medium; but its very inelasticity, while freeing it from loss of power in one direction, only exposes it to loss in another, and not improbably to an equal extent. The motive effort of water-power is obviously invariable, whatever resistance may be opposed to it, and, consequently, the same expenditure of energy is necessary whether the work done be considerable or insignificant.

On the other hand, hydraulic machinery, when working at full power, is characterised by a high efficiency; the loss due to the friction of the working parts then rarely exceeds 8 or 10 per cent.* Furthermore, there is great smoothness and regularity of movement, and the appliances are capable of being manipulated with extreme precision, while they do not call for specially trained or skilled operators.

As against this, must be set the trouble and inconvenience caused by frost. Apart from the freezing of water in the conduits, which in many cases are unavoidably exposed to atmospheric influence, there is the consideration that the neighbourhood of hydraulic machinery is invariably wet and sloppy, and this leads to the formation of ice there, which is manifestly dangerous to those working at a quay side. The evils have to a certain extent been mitigated by the provision of gas jets in machinery pits, or by bringing all the service pipes and valves into a closed cabin which can be artificially warmed when necessary. But such arrangements, whilst more or less effective in themselves, are undoubted evidence of the difficulties attending the use of water-pressure machinery in the winter

* Mr. Robinson gives the following coefficients for hydraulic rams with ordinary hemp packing:—

Direct-acting,	·93 efficiency.
2 to 1,	·8 ,,
4 ,, 1,	·76 ,,
6 ,, 1,	·72 ,,
8 ,, 1,	·67 ,,
10 ,, 1,	·63 ,,

"Transmission of Power," *Min. Proc. Inst. C.E.*, vol. xlix.

time; and in countries where the thermometer is often below zero, it would be difficult to secure perfect immunity from interruption of working.

To this drawback must be added the great cost of laying mains and forming culverts for their reception. Water pressure is also very materially affected by bends and changes of direction, so that where these are inevitable there will be a corresponding loss of power.

Electricity.—As a distributive agent, electricity has very largely come into favour during the last ten to fifteen years. Its principal merits are the extreme cleanliness and compactness of its working parts, and the tenuity and flexibility of its supply mains, both of which features stand out in prominent juxtaposition to the soot and smoke of the steam engine and the bulky and awkward canalisation of hydraulic power. Moreover, the first cost of wire mains is much less than that of any corresponding pipe system.

As a motive force, electricity is able to discharge all the functions of steam for actuating mechanism identical in character. The main shaft of a machine may be driven indifferently by a steam engine or an electric motor. But whereas steam power is rarely capable of centralisation, electricity is most admirably adapted to systematic distribution from a common centre. Since steam is most commonly employed for the generation of the electric current itself, it is not contended that the latter system is as economical as the former; but it may be pointed out that one large electrical generating station, worked by steam power, will probably involve less expenditure in fuel, repairs, maintenance, and attendance than a number of separate steam engines, each with its own special outfit and upkeep. Moreover, the generating plant may find an additional use at night-time for lighting purposes, and this at a period when, lifting appliances being more or less idle, there would be little or no interference with the discharge of its primary functions. There must inevitably be considerable saving arising from the adaptation of a single installation to the supply of both power and light. The advantages arising from the combination are, however, largely discounted during the winter, when the shortness of the days necessitates early lighting.

The amount of electrical energy consumed is sensibly proportional to the work done, and in this respect electric power differs advantageously from hydraulic power.

Electric distributors, however, are more complicated than the working parts of the other two systems; they are, therefore, less easily kept in repair, and they necessitate the attendance of skilled workmen. Moreover, they do not act with the smoothness and precision of hydraulic machines, nor with the independence and directness of the steam engine.

Comparative Expenditure of Energy.—In order to institute a comparison between the several systems in regard to their expenditure of energy and the cost of its production, it is necessary to establish the relationship existing between their respective units of power. The primary unit of work is the

foot-pound, and 33,000 foot-pounds per minute constitute 1 horse-power, the basis upon which steam-engine power is estimated. Hydraulic power is specified in terms of the supply of gallons of water per hour at a definite pressure, and electric power in Board of Trade units. To trace the connection between the various standards we proceed as follows :—

If H be the head in feet of a column of water and P its pressure per square foot,

$$P = w\,H,$$

where w is the weight of a cubic foot of water.

Consequently the pressure per square inch is

$$p = \frac{w\,H}{144};$$

and, taking w at 62·5 lbs., $H = 2{\cdot}307\,p$.

Hydraulic pressure of x lbs. per square inch is therefore that due to a head of 2·307 x feet, and the potential energy of, say, 1,000 gallons at this head is

1,000 gallons × 10 lbs. per gallon × 2·307 x feet = 23,070 x foot-lbs.

Assuming this to be the hourly rate of supply, and noting that 1 horse-power hour is equivalent to 33,000 × 60 = 1,980,000 foot-lbs., we conclude that 1,000 gallons of water at x lbs. pressure possess energy to the extent of $\frac{23{,}070\,x}{1{,}980{,}000} = {\cdot}01165\,x$ horse-power hours.

The Board of Trade unit of electricity is 1,000 watts per hour. The watt is the product of 1 volt (the unit of head or pressure) into 1 ampère (the unit of current), and corresponds in electrical terminology to the foot-pound of mechanics; 746 watts are equivalent to 1 electrical horse-power. Therefore, at the same rate of supply, the Board of Trade unit $= \frac{1{,}000}{746} = 1{\cdot}34$ horse-power hours.

From the foregoing data the following table is deducible :—

TABLE XXXVII.—Comparison of Power Supply.

One Thousand Gallons of Water under the Following Pressures :—	Equivalent to Energy in	
	Horse-Power Units.	Board of Trade Electrical Units.
700 lbs. per square inch,	8·16	6·09
750 ,, ,,	8·74	6·52
800 ,, ,,	9·32	6·95
850 ,, ,,	9·90	7·39
900 ,, ,,	10·48	7·82
950 ,, ,,	11·07	8·25
1000 ,, ,,	11·65	8·69
1250 ,, ,,	14·56	10·86
1500 ,, ,,	17·48	13·04

Crane Tests.—Tests with cranes afford a convenient and the most practicable criterion of power expenditure,* and some very interesting experiments have been made in this connection by M. Delachanal, the engineer to the Havre Chamber of Commerce, the results of which are tabulated below. The operations, which were carried out at the port of Havre, consisted in the lifting of loads of 29·5 cwts. (1,500 kilogrammes) and 7·88 cwts. (400 kilogrammes), respectively, by each crane to a height of 29·5 feet (9 metres), at which point the crane was slewed through an angle of 180° and the load lowered and deposited. The empty hook was then raised to the same height, steered through a semicircle in the inverse direction, and lowered for a fresh load. In the case of electrical power, variation in speed was effected by a rheostat in series with the motor.

The actual working expenditure per hour is given by multiplying the tabular figures by 30, 40, or 50, according to the rate of working.

TABLE XXXVIII.—Expenditure of Energy by Cranes in Foot-lbs. per Operation.

Speed in Feet per Sec.	Duration of Lift in Secs.	Hydraulic Crane. Load.		Steam Crane. Load.		Electric Crane. Load.	
		29·5 Cwts.	7·88 Cwts.	29·5 Cwts.	7·88 Cwts.	29·5 Cwts.	7·88 Cwts.
·492	60	318,975	190,589	458,543	359,263	913,332	564,600
1·312	22·5	318,975	190,589	496,053	380,860	411,868	278,998
2·132	13·85	318,975	190,589	566,054	392,525	295,656	212,049

The work effectively performed in each case was 97,645 ft.-lbs. and 26,038 ft.-lbs. respectively.

The figures demonstrate the disadvantage of making steam cranes work too quickly and electric cranes (with series wound motors) too slowly. At the higher and more usual speeds, steam cranes are shown to be much inferior to hydraulic and electric cranes.

Equally interesting experiments have been restricted to a comparison of these last two agencies. Thus, Mr. Philip Dawson † has recorded the following expenditure of power in watt-hours for hydraulic and electric cranes under similar conditions of working. The cycle of operations consisted of a lift of 36·1 feet, a slew of 140°, and a lower of 13 feet, all under load, with the inverse movements unloaded. The hydraulic crane had three powers.

* It is difficult in the case of other apparatus to obtain identical conditions for the purpose of experiment.

† *Traction and Transmission*, May, 1903.

TABLE XXXIX.—EXPENDITURE OF POWER IN WATT-HOURS PER CYCLE.

	LOAD.				
	½ Ton.	¾ Ton.	1 Ton.	1¼ Ton.	1½ Ton.
Hydraulic crane, . .	82·0	127·2	127·2	172·4	172·4
Electric ,, . .	48·5	58·5	73·5	80·5	105·5

A test with two cranes at Glasgow, carried out by Mr. Baxter of the Clyde Navigation, was analysed by Mr. Walter Pitt * in the same units. The lift in this case was 30 feet, the slew 100°, and the lower 10 feet. The hydraulic crane had only one power, and consequently was at a great disadvantage in regard to the lighter loads. At its full load it exhibited a superiority to the electric crane.

TABLE XL.—EXPENDITURE OF POWER IN WATT-HOURS PER CYCLE.

	LOAD.			
	1 Ton.	2 Tons.	2½ Tons.	3 Tons.
Hydraulic crane, ...	236·7	236·7	236·7	236·7
Electric ,, ...	83·3	160·4	197·9	241·9

Cost of Power.—Greater expenditure of energy does not necessarily involve a correspondingly greater cost of working. This, of course, depends on the relative rates at which power can be supplied, and will vary with different localities. Equal conditions prevail when the cost of water under pressure bears to the cost of electricity the ratios given in Table xxxvii. Thus, electricity at 3d. per Board of Trade unit is the equivalent of water under 750 lbs. pressure at $3 \times 6{\cdot}52 =$ 1s. 7½d. per 1,000 gallons.

A comparison of the cost of hydraulic power and electric supply as compiled by Mr. Ellington † from the Reports of the London Hydraulic Power Supply Company (L.H.P.), and the Westminster Electric Supply Corporation (W.E.S.), for the year 1894, yielded the following results :—

* Pitt on "The Modern Equipment of Docks," *Eng. Conf.*, London, 1903.

† Ellington, "Notes on Hydraulic Supply in Towns," *Proc. I. Mech. E.*, July, 1895.

TABLE XLI.

1894.	Total Amounts.		Comparison in Gallons at 750 Lbs. Pressure per Square Inch.		Comparison in Board of Trade Electrical Units.	
	L. H. P.	W. E. S.	L. H. P.	W. E. S.	L. H. P.	W. E. S.
	£	£	Gals.	Equivalent Gals.	Equivalent Elec. Units.	Elec. Units.
Capital outlay,	471,552	411,018	...	...	...	...
Output,	...	...	400,313,000	396,256,000	2,609,240	2,582,801
Quantity sold,	...	...	332,390,000	333,430,000	2,166,520	2,173,298
Received for supply,	49,237	50,729	...	...	...	...
Average price obtained,	...	...	35·55d. per 1,000 gals.	36·51d. per 1,000 gals.	5·45d. per unit.	5·6d. per unit.

The actual cost of production, or station cost, was 5·17d. per 1,000 gallons of water and 1·38d. per electrical unit. Both power supplies can now be obtained at a much cheaper rate. At the present time the total cost of electricity at the switchboard, amounts to ·9d. per unit at Liverpool and to only ·35d. per unit at Newcastle, the cost of coal being, no doubt, responsible for the difference. The station cost of hydraulic power at London, in 1900, was given as 3·03d. Electric power is furnished to consumers at 1d. per unit at Wigan and at 1½d. per unit at several other towns, including Liverpool, at which last named place the price of hydraulic power (750 lbs. pressure) ranges from 15d. upward.

Conclusions.—Reviewing the systems as a whole, the precedence will be generally accorded to electrical energy for convenience and adaptability, and to hydraulic energy for simplicity and control. Where a hydraulic installation is already in existence, a change to an electrical régime could scarcely be justified in this country on other than the most exceptional grounds; but where the question is an open one and unfettered by conditions, there is a slight preponderance of evidence in favour of the adoption of electricity for the transmission of power.

At all events the two systems are in such general vogue—either singly or in combination—at nearly all ports as to merit some discussion in regard to the lines of their application and their suitability for particular classes of work.

Hydraulic Machinery.—The development of hydraulic power constitutes one of the most remarkable features of the past century. From a comparatively insignificant position, as a source of energy, water pressure suddenly and rapidly rose to a foremost place in engineering operations. Any attempt at tracing the inception and expansion of water-pressure machinery would, however, necessitate a lengthy retrogression into history, and this we cannot afford here. But it will be generally admitted that, apart from the Bramah press, the present wide range of useful applications for water power is mainly due to the ingenuity and the exertions of the late Lord Armstrong. The student who is interested in the historical aspect of

the subject will find much entertaining and valuable information contained in a paper read by him before the Institution of Civil Engineers in 1877.*

The modern hydraulic machine (for dock work) takes the form either of a direct-acting ram, working backwards and forwards in a cylinder with suitable multiplying gear for increasing the effective length of its stroke, or of a bent crank with rotary motion imparted by two or more pistons also working in cylinders. The former system is most commonly applied to gate and sluicing machines, and to cranes; the latter, generally, to capstans, and occasionally to gate machines.

We will deal, first of all, with the ram apparatus. Primarily, this consisted of a ram fitting into the bore of a cylinder, the pressure being applied at one end of the ram, so that it was, accordingly, capable of acting in a forward direction only. The return stroke, being unopposed, was effected either by gravitation, if the ram were vertical, or by a small auxiliary ram, if the main ram were horizontal. One important drawback of this contrivance was that it admitted of no variation in the power applied. Whether the load moved were great or small, the same expenditure of energy was necessary. When the load was fairly uniform, as in the working of dock

Fig. 516.—Combined Piston and Ram.

gates and sluices, the objection was of little importance, and this type of machine is still largely used for that purpose. But in the case of cranes and other lifting apparatus, where loads are irregular, economy demands some modification so as to make the expenditure of water correspond approximately to the actual load. This has been contrived by the use of two or three cylinders, able to act either independently or collectively. Three power values have, however, been found superfluous, or, at anyrate, unduly cumbersome in practice, and it is now customary to be satisfied with two powers at the most, and these are obtained with a single cylinder in one of two different ways:—

First, by the use of a combined piston and ram (fig. 516), water being admitted to both sides of the piston for the lower power, and to the larger side only for the higher power. This arrangement is now very rarely used, one of the reasons being that a bored cylinder is required, the machined surface of which becomes corroded while out of action, with the result that the packing on the piston is cut.

Secondly, by the use of two concentric rams (fig. 517), one contained within the other, in the same cylinder. For the lower power, the smaller

* Armstrong on "Water-pressure Machinery," *Min. Proc. Inst., C.E.*, vol. l.

ram only is put in motion; for the higher power, the larger ram is liberated and moves simultaneously with it.

In some instances, a combination of both the preceding methods has been utilised to obtain three powers from one cylinder. The outer or larger ram is fitted with a piston so as to give two powers by the first method, while the internal ram supplies the lowest power. The arrangement is, however, so complicated as to be of doubtful utility, and, except in extreme cases, it will be found preferable in this respect to sacrifice economy to efficiency. The uniform expenditure of water upon work of the most variable nature cannot be considered excessive when it is borne in mind that simplicity in construction and manipulation has advantages to offer nearly, if not quite equivalent, to economy in power.

The second system is based on the principle of the reciprocating action of the connecting-rod and crank-shaft of the ordinary steam engine, and one type of the apparatus consists of three small cylinders with plungers, each acting upon a three-throw crank and having mitre valves, worked by cams upon a revolving shaft.

Fig. 517.—Two Concentric Rams.

Another type (fig. 534), until recently much in vogue, had only two cylinders. These oscillated upon trunnion bearings, and were fitted with combined rams and pistons working on over-end cranks set at right angles to each other. The areas exposed to pressure in the cylinder were as two to one. One face of the piston had the exact moiety of the area of the other face, the difference being due to the displacement of the ram. The pressure on the smaller face was maintained constant, there being continuous communication with the supply pipe. The pressure on the larger face was intermittent, and alternately full and nil, according as the cylinder on that side was open to supply or exhaust. The piston, accordingly, was actuated by the difference of pressure on its two faces, the stroke in one direction being effected by unopposed pressure on the smaller face, and in the other direction by the balance of pressure on the larger face, which, by the adjustment of areas, resulted in regularity of effort.

The three-crank system is adapted for large engines in situations where there is ample space at disposal. The two-crank system, on the other hand, is more compact and also less expensive in construction, in that a middle crank is obviated, but it lacks the uniformity of movement, characteristic of

the former arrangement. Furthermore, the saving effected by omitting one cylinder and ram is largely discounted by the cost of making the other two cylinders double-acting, and almost necessarily of brass. The maintenance charges also are greater. Except, therefore, in the case of restricted space, the three-cylinder system with plain rams is generally adopted.

We now turn our attention for a moment to the production of hydraulic power.

Hydraulic Accumulators.—In the first instance the requisite pressure for driving hydraulic machinery was obtained by means of a natural head of water, but this system, in the majority of cases, the locality being flat, involved the erection of a lofty water tower and reservoir. The impossibility of economically erecting such a tower at New Holland on the Humber, where the foundation consists of silt to a considerable depth, led Armstrong, in 1850, to substitute an arrangement, since generally adopted and known as an "accumulator," by which water was pumped into a large cylinder against the weight of a heavily loaded ram or plunger. As long as the ram is kept off its seat at the bottom of the cylinder the water is maintained at a high and constant pressure—at a much higher pressure, in fact, than could be obtained by natural means; for, whereas before the introduction of the accumulator, in no instance had a greater pressure than 90 lbs. per square inch been used, at the present time pressures as great as 700 and 750 lbs. per square inch are quite common, and 1,000 and 1,250 lbs. pressures are also in use. The advantages arising from this increment are apparent. The sizes of the distributing mains and of the pressure cylinders have been greatly reduced, while at the same time the capacity for work has been materially augmented. The accumulator has one drawback: it does not afford much storage room, consequently pumping is necessarily continuous, and the joints and pipes in the mains must be rendered pressure proof. These considerations, however, are of minor importance compared with the advantages accruing to the system as a whole.

It is essential that the water used in connection with hydraulic apparatus should be both fresh and clean. Salt or acidulated water will corrode the mains and cylinders; grit and sediment will wear and choke the valves. Consequently, where the source is at all liable to contamination there should be a settling tank, and supplies should be taken from the top in such a way as to ensure purity. There is no objection to the repeated use of the same water; in fact, this arrangement is generally adopted, the water being returned to the pumping well through an additional main, the diameter of which is rather greater than that of the pressure main.

Slide valves are more liable to injury from grit than mitre valves, but if the settling tank be adopted and ordinary precautions observed, there is no reason why extensive repairs should be necessary in either case.

Air vessels have been tried in place of weighted accumulators, but they

are open to the objections that the pressure is by no means constant, that the storage is generally insufficient, and that, in some instances, there is loss arising from the absorption of air by the water, which has to be replaced by an auxiliary feed-pump. There are situations, however, such as on board ship, where accumulators are inadmissible and where air vessels have the advantage of lightness.

Fluctuations in Pressure.—Hydraulic power in application to dock-work is liable to extreme changes in amount. The constantly varying number of machines under action, while the area of the supply main is always the same, causes the intensity of pressure to fluctuate considerably. It frequently falls much below the nominal value, and sometimes, under the influence of surging, it may rise above it. The following readings, recently taken in connection with the working of the entrance gates to a dock at Liverpool, illustrate this irregularity very forcibly :—

TABLE XLII.

Time.	Locality.	Draught of Water on Sill.	Maximum Pressure Prior to Movement of Ram.	Working Pressure fairly Constant throughout Stroke.
A.M.		Feet Ins.	Lbs.	Lbs.
6.50	80 feet entrance,	30 0	730	680
6.58	40 ,,	25 0	730	730
7.0	100 ,,	29 0	730	530
9.0	80 ,,	21 0	730	690
9.10	40 ,,	16 3	750	350
9.15	100 ,,	20 6	760	560
11.0	80 ,,	15 6	750	720
11.5	40 ,,	11 0	750	300
11.10	100 ,,	15 0	760	600
P.M.				
1.0	80 ,,	16 6	740	720
1.10	40 ,,	12 0	760	260
1.20	100 ,,	17 6	760	580

The normal pressure was 750 lbs. at the accumulator. The areas of the rams were as follows:—80-feet entrance, 227 square inches; 40-feet entrance, 113 square inches; 100-feet entrance, 283·5 square inches.

Electrical Distribution of Energy.—Electricity, as a practical science, is much the junior of hydraulics, and, in reference to dockwork, it has only been adopted to any noticeable extent within the last decade. Hamburg and the German ports of the Baltic introduced it about the year 1892. It was speedily taken up by Rotterdam, Amsterdam, Bordeaux, Havre, and Copenhagen. Southampton is apparently the first port at which it appeared in this country, but the use of electricity is now rapidly becoming general and, where the question is not complicated by the prior existence of a hydraulic installation, its claims for selection are admittedly pre-eminent.

The electric current is either continuous or alternating, and this latter case either single or multiphase.

The continuous current flows uninterruptedly in one direction, being the reverse of the alternating current which flows alternately in opposite directions. Multiphase currents are a group of the latter type which differ from each other by their relative difference in phase.

The continuous current has hitherto proved to be the most satisfactory for dealing with operations so variable in nature as those which prevail in connection with dockwork. Alternating single-phase currents only give good results when utilised at a fairly uniform speed, and without the necessity of overcoming the inertia of heavy bodies at starting. Continuous currents, on the other hand, on account of insulation difficulties with regard to the construction of both armatures and commutators of the generators, and the fact that it is necessary to use rotating transformers for reducing the pressure, are not so well adapted for the transmission of power to very long distances, though within the ordinary limits of most dock systems, they will be found perfectly effective and sufficient.

The dynamos and motors generally utilised may be enumerated as

(1) Series wound,
(2) Shunt wound, and
(3) Compound wound.

In the first case, the armature, the field winding, and the external circuit are all in series. In the event of short circuiting, the field current is intensified and the winding may be injured by the heating of the wire.

In the second case, the field winding is distinct from the outer circuit, and there is, consequently, a separate current to excite the field magnets. Short circuiting can, therefore, produce no heating effect.

The compound machine has two coils on its field magnet. One winding is in series with the external circuit and the armature, the other is in shunt. This machine, from the counter action of its coils, is more regular under the influence of varying currents than either of the other two, but it is only completely regular and automatic at one particular speed.

A series-wound motor is suitable for use in positions where great starting power is required, such as in cranes, haulage gear, &c., and also in the case of single motors, driving pumps, and heavy machinery where the load is constant after being once applied. When run off constant pressure circuits, the motors are controlled by a variable resistance placed in series with them and regulated by hand, as required. In the series-wound motor the speed decreases as the current increases. The torque is greatest at starting when the current is a maximum, being about six times the normal amount, and, as it is proportional to the latter, it varies inversely as the speed. When the load varies the speed is not constant.

The shunt-wound motor is not so well adapted for starting against a heavy torque as the series-wound motor. It will, however, run at nearly constant speed under a varying load when supplied with current at constant pressure. With the shunt motor, also, considerable variation in speed can

be obtained by varying the resistance in the shunt circuit, and so affecting the exciting current. The starting torque is about three times the normal amount.

Compound-wound motors may have their field coils wound either differentially, with the series coils in opposition to the shunt coils, or cumulatively, with the series coils assisting the shunt coils. Where great regularity of speed is required the differentially-wound motor is probably the better, but it has not met with any great measure of success. One objection to it is the liability to start in the wrong direction, owing to the reversed series winding. The most important feature of the cumulatively-wound motor is the increased torque at starting, due to the series coil. It combines, in fact, to a certain extent, the starting power of the series motor with the speed regulation of the shunt motor. In this last respect, however, it is not so good as the shunt motor. This type of motor is sometimes fitted to cranes where the motor is allowed to run constantly, and in such situations has given good results.

Applications of Power.

The various types of appliances, which it is proposed to briefly describe, may be classified under the following heads :—

Gate machinery,
Sluice machinery,
Capstans,
Wharf and floating cranes,
Jiggers and transporters,
Slipway machinery,
Coal tips and hoists,
Grain elevators,
Pumps,
Miscellaneous apparatus, such as moorings, &c.

Dock Gate Machinery.—Dock gates may be worked by means of chains or of arms or struts.

The chains may be wound on barrels or drums in gear with rotary shafts driven by steam, hydraulic or electric power, or they may pass over sheaves at the ends of the cylinder and ram respectively of a hydraulic machine. An example of the former class is that given in figs. 518 and 519, which show the plan and section of a gate crab or winch worked by hydraulic power. The ram system has already been exemplified in figs. 516 and 517. Where space is restricted and long chains are necessary, a cupped drum grasping the links of the chain will be used in preference to a barrel, which is less compact. There are drawbacks, however, to this arrangement, in that special links are required, and that a corresponding adjustment must be made whenever stretching occurs.

Chains, in fact, call for constant attention. They must be frequently

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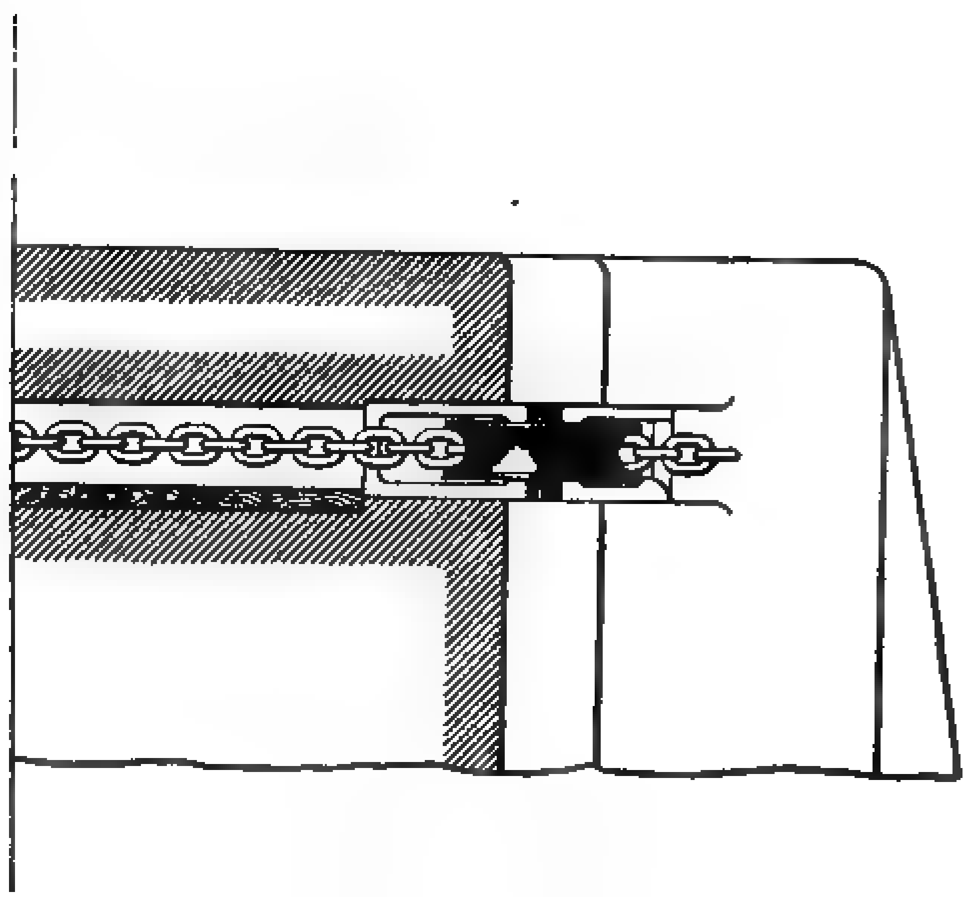


overhauled and examined, and should be annealed in a wood fire at least once a year. This involves the provision of spare chains, and these should be ready for instant substitution, in case of breakage or other serious accident. The advisability of the chain connections being simple and accessible is therefore apparent. For gate attachments below water level a ring at the end of the chain, of larger diameter than the staple through which the chain passes, will be found a suitable arrangement.

The expansive system of the hydraulic ram with multiplying sheaves seems, on the whole, preferable to the rotary engine, owing to the greater risk of damage to the gearing of the latter. Damage to chains and mechanism arises principally from such causes as irregularity of movement, with abrupt jerks and stoppages, which induce momentary stresses of unexpected magnitude. Fracture or strain may easily result from an attempt to force a gate home in the face of some submerged obstruction, and as it is preferable for a gate to be brought to, rather than for a breakage to occur, it is by no means judicious to provide machinery of excessive power, unless it be carefully regulated.

Gate chains are arranged on the two systems indicated in figs. 520 and 521. In the first case, chains are attached to the back and front of the gate respectively, near the bottom and, being led horizontally to sheaves set in the walls, at opposite sides of the passage, they pass vertically upwards to other sheaves near the coping level, whence they are conducted to their respective machines. In the second system, known as the "overgate system" (fig. 521), chains (A and B) are fixed to the opposite walls of the passage and led horizontally to sheaves at the foot (C) of the gate, thence vertically upward to sheaves at the top of the gate, and, finally, in a parallel course, over a third pair of sheaves near the heel-post to the actuating gear. By this latter arrangement, each leaf of the gate is opened and closed from the same side of the passage and from one spot. Thus, the cost and inconvenience of two separate chain-ways through the walls to the machine pits are avoided.

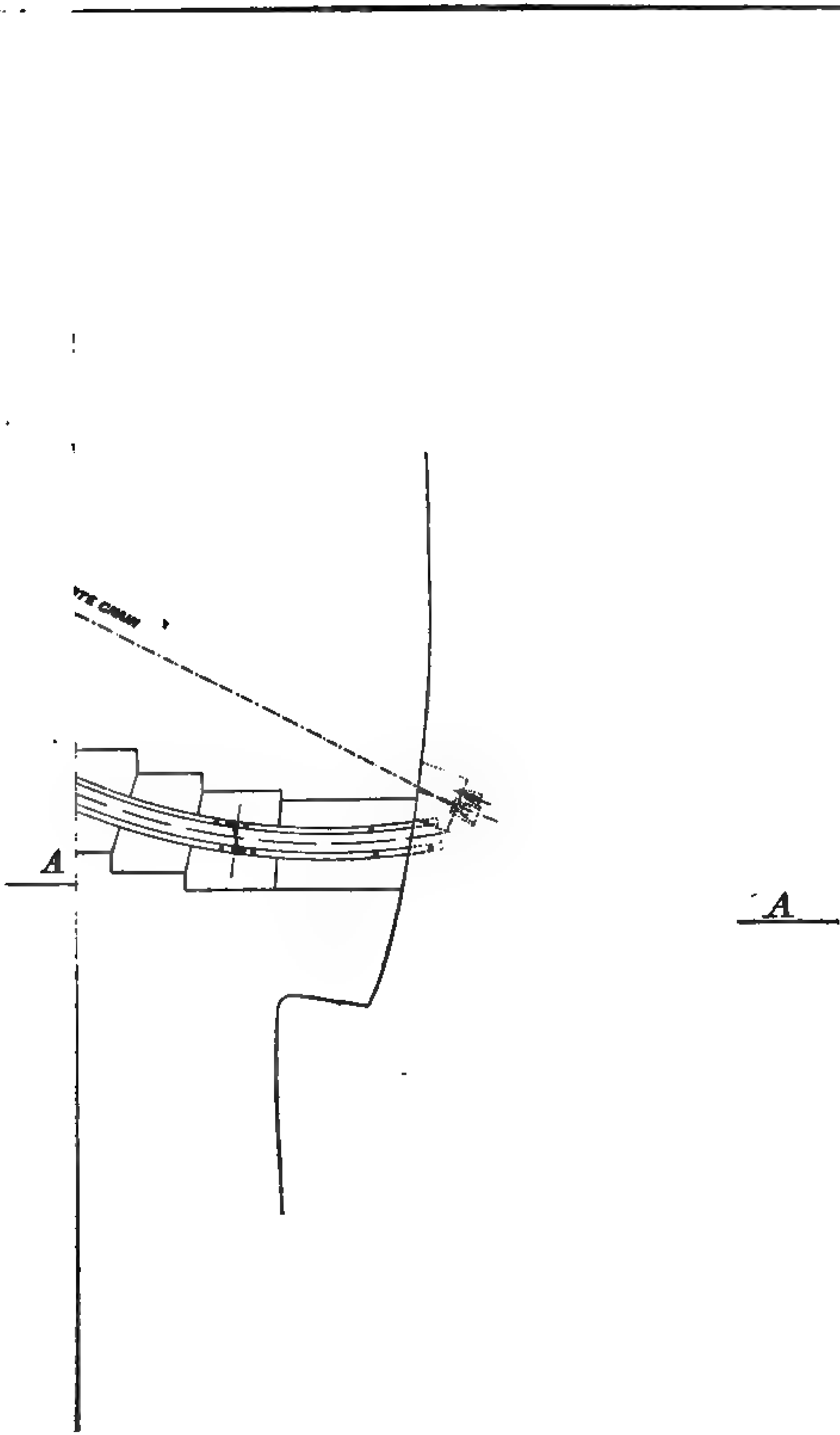
Struts or direct-acting rams were introduced by Sir J. W. Barry for working the gates at the Barry Docks in 1894. They have the possible advantage over chains of being able to hold the gate up against external pressure, and thus discharge the functions of a strut gate in minimising the effect of waves at high water. This advantage, however, is more apparent than real, as the power of gate machines, unless unduly great, is inadequate to do more than work gates under ordinary circumstances. The earliest examples of direct-acting rams worked in cylinders oscillating upon trunnions, but this type has not been repeated, at all events in this country. Recent practice has entirely favoured a fixed cylinder, with ram and connecting-rod, which latter, by means of a crosshead and vertical and horizontal pivot pins, is free to turn in any direction. The gates at Leith, illustrated in figs. 526 and 527, are worked in this manner, as also are the West India Dock gates at London, and many others.

The ram or the connecting-rod, as the case may be, is usually attached to the gate through the medium of a girder or radius arm, one end of which is fixed to the heel-post and the other to a point somewhere about one-third of the length of the leaf from the mitre-post. In this way the pressure of the ram is more effectively applied to the gate, but the fact that the application of pressure is necessarily above the water line militates against any com-

Fig. 520.—Arrangement of Gate Chains—Direct System.

Fig 521.—Overgate System.

pletely satisfactory arrangement. The objection has possibly not so much weight in regard to iron and steel gates, which can be suitably stiffened, but is of so great importance to wooden gates as to have rendered the system practically inapplicable to such gates, owing to the difficulty of making them sufficiently rigid. In some cases attachment is made at a point more distant from the heel-post, and the stroke of the ram is correspondingly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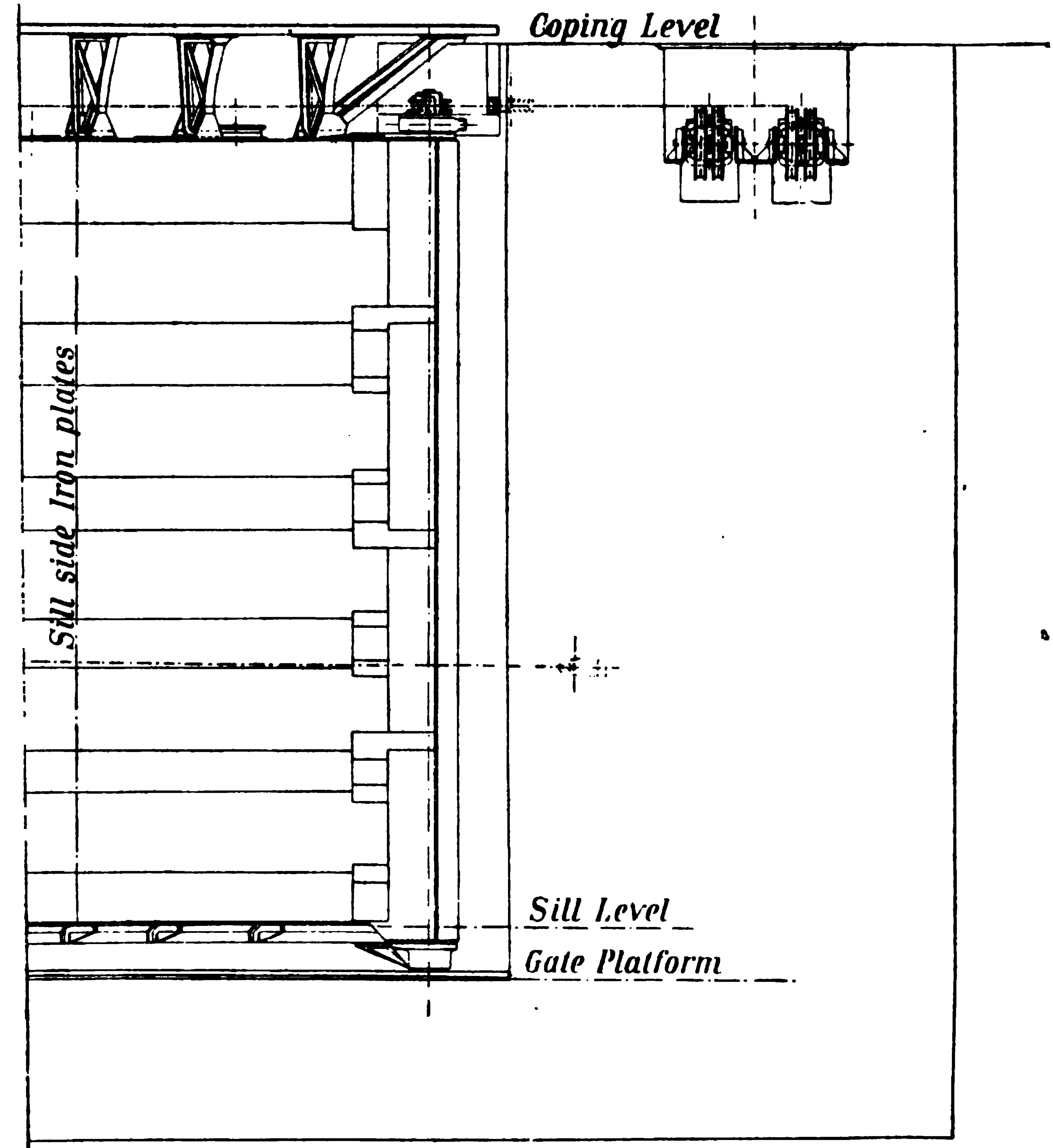

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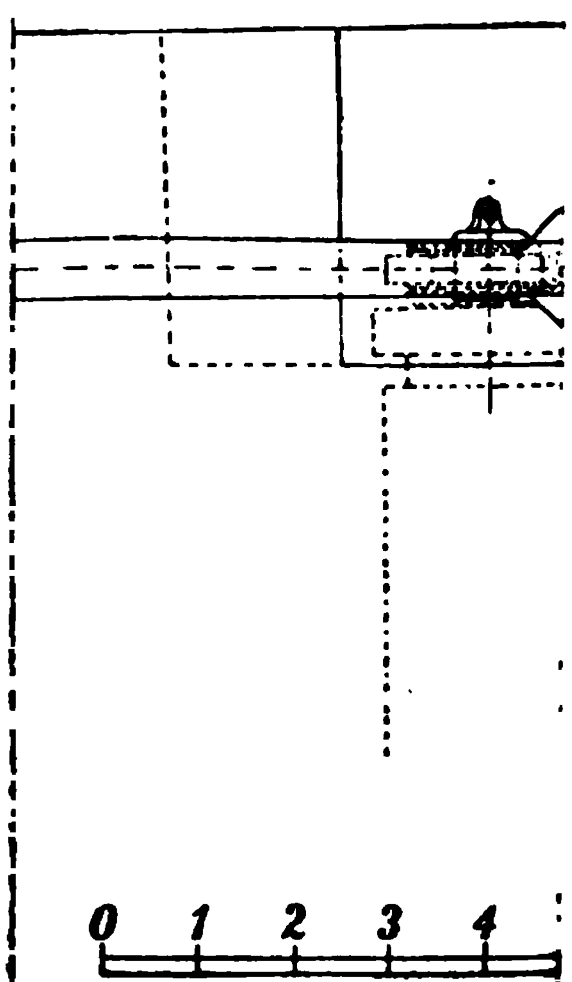

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increased. The chain system is preferable to the ram in this respect, for it is quite feasible to attach the chains at the centre of gravity of the displaced fluid, which is the ideal position.

One advantage which the rotary engine possesses over the ram is that in certain cases of breakdown—viz., those not involving the gearing, barrel or chain—the former can still be worked by hand, whereas it is never possible to work a ram in this way. The expedient then generally resorted to is to attach a rope to the head of the mitre-post of the gate and lead it to the nearest capstan. This is by no means a desirable or convenient arrangement, but it should nevertheless be looked upon as a likely contingency and provided for accordingly.

A point which must not be overlooked is that chains reduce the effective draught of water over dock sills, and that, in order to allow the former to lie perfectly flat, so that vessels passing over them may not foul, it is necessary to provide a large amount of slack chain. Chases have been cut in the sill to receive the chain, but it is by no means certain that the latter will lie in them.

The accompanying illustrations (figs. 522 to 527), showing the application of hydraulic power, by means of both chains and rams, to recently constructed gates at Leith, are reproduced from drawings kindly furnished by Messrs. Sir W. G. Armstrong, Whitworth & Co., with the courteous sanction of Mr. Peter Whyte, the harbour engineer of that port.

Power of Gate Machines.—While the determination of the amount of power necessary to work cranes, capstans, and other dock appliances is a matter of comparatively simple calculation, the paucity of existing data in reference to the forces at work upon dock gates renders the problem in this last instance apparently incapable of an exact or, at any rate, a general solution. There can be little doubt that, in the majority of cases, a large margin of power has to be provided to cover unknown contingencies.

The resistances to be overcome are three in number. At the moment of starting there is the inertia of the gate, and during movement there are the friction of pintles, collars, wheels, rollers, &c., as the case may be, and the resistance of the water to disturbance by the motion of the gate.

The force required to overcome the first of these may be estimated as follows:—Calculate the moment of inertia of the gate about its axis of rotation; for the purpose it may be treated, without serious error, as a weighted rectangle revolving about one edge. Then

$$I = \tfrac{1}{3} M l^2,$$

where M is the mass of the leaf and l its length. From this we find the radius of gyration, which is

$$\sqrt{\frac{I}{M}} = \sqrt{\frac{1}{3} l^2} = \cdot 577\, l,$$

and the mass of the leaf may accordingly be considered concentrated at a point distant $\cdot 577\, l$ from the axis of rotation.

If the gate chain be attached at a distance, x, from the same origin, the equivalent mass on the line of pull is

$$\frac{\text{M} \times \cdot 577}{x} = m, \text{ say.}$$

Now, if it be desired to impart to such a mass a velocity of v feet per second in, say, t seconds, the acceleration will be $\frac{v}{t}$, and the force, f_1, required to produce it

$$f_1 = \frac{m}{g} \times \frac{v}{t}. \quad . \quad . \quad . \quad . \quad (136)$$

The second force, f_2, which is required to overcome friction, may be estimated with the aid of a suitable coefficient, c, at some fraction of the weight (W) to be moved.

$$f_2 = c\,\text{W}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (137)$$

Lastly, the resistance of the water to displacement during movement is theoretically determined by the consideration that the pressure on the plane of the gate is some factor of the pressure which would be produced by a body of water falling upon the gate with a velocity equal to the velocity of movement of the latter.

If h be the distance through which the water is supposed to fall, we have, in lbs.,

$$\begin{aligned} f_3 &= \text{A} \,.\, wh \,.\, k \\ &= \text{A} \,.\, \frac{w v^2}{2g} \,.\, k \quad . \quad . \quad . \quad . \quad (138) \end{aligned}$$

where, for fresh water, $w = 62\frac{1}{2}$ lbs. and $k = 1{\cdot}8$; and for salt water, $w = 64$ lbs. and $k = 1{\cdot}85$. A is, of course, the area in feet of one surface of the leaf.

As an example let us take the case of a greenheart gate, 55 feet long by 40 feet deep, with the possibility of the full extent of head, and suppose it to be worked by a chain attached at a point 10 feet from the outer extremity of the leaf. Assume the weight of the gate to be 150 tons. Then

$$m = \frac{150 \times \cdot 577 \times 55}{45} = 105{\cdot}8 \text{ tons};$$

and if it be deemed desirable to obtain a speed of 1 foot per second, in ten seconds,

$$f = \frac{105{\cdot}8}{32} \times \tfrac{1}{10} = \cdot 33 \text{ ton.}$$

This is on the supposition that the pull is horizontal; any deviation therefrom would necessitate a suitable modification.

The frictional coefficient is most difficult to estimate in the case of a dock gate, there being so many modifying influences at work. For a railway train travelling at normal speed about 10 lbs. per ton would be considered a fair allowance, but this coefficient is manifestly too low for a cumbersome

greenheart gate, moving at a much slower rate with conical rollers over splayed tracks, and it will still further be augmented by a certain amount of friction at the heel-post. On the other hand, there is the diminution of the load on the rollers due to flotation, which will, of course, vary with the depth of water at the time of working. Further, there is the ratio of the diameter of the roller to that of its axle, and the proportion of weight which the roller carries. With a ratio of 4 to 1 and a coefficient of ·15, the friction due to that portion of the gate borne by the roller would be $\frac{2,240 \times \cdot 15}{4} = 84$ lbs. per ton. Allowing for flotation and dealing with the question, as is inevitable, in a somewhat rough and ready way, it will probably not prove an excessive estimate if we take the frictional resistance of the gate at 20 lbs. per ton on its gross weight, in which case

$$f_2 = \frac{150 \times 20}{2,240} = 1 \cdot 33 \text{ tons.}$$

For the resistance offered by salt water to displacement we have

$$f_3 = \frac{55 \times 40 \times 64 \times 1 \cdot 85}{64 \times 2,240} = 1 \cdot 81 \text{ tons.}$$

Hence the maximum tension in the chain, exerted at the moment of starting the movement of the gate, will be

$$T = f_1 + f_2 + f_3 = 3 \cdot 47 \text{ tons.}$$

This figure will need some additional margin to cover uncertainties in the frictional resistance. Under circumstances only too common in connection with the working of dock gates, the resistance may easily be increased to double the amount calculated above, for which fair conditions of track have been assumed.

The following table exhibits data relating to several existing examples of machinery for greenheart gates. For metal gates with buoyancy chambers the friction of movement will be much less, and the amount of power to be applied will accordingly be considerably reduced.

TABLE XLIII.—GATE MACHINES.

Width of Entrance.	Greatest Working Head.	Least Working Head.	Area of Surface of Leaf.	Diameter of Ram of Gate Machine.	Diameter of Chain.	Gear.	Accumulator Pressure.
Feet.	Feet.	Feet.	Square Feet.	Inches.	Inches.		Lbs.
40	36·5	26·5	766·5	12	1	6 to 1	750
80	41	31	1722	17	$1\frac{3}{16}$	6 ,, 1	750
80	40	30	1680	17	$1\frac{3}{16}$	8 ,, 1	850
90	41	31	2091	18	$1\frac{3}{8}$	6 ,, 1	750
94	34	24	1768	18	$1\frac{3}{8}$	6 ,, 1	750
100	41	31	2316·5	19	$1\frac{1}{2}$	6 ,, 1	750
100	40	30	2260	19	$1\frac{1}{2}$	6 ,, 1	850
100	39	29	2203·5	20	$1\frac{1}{2}$	6 ,, 1	1000

Figs. 528 and 529.—Electric Clough at Ymuiden Locks.

Sluicing Machinery.—The penstocks or cloughs which regulate levelling and sluicing culverts may be worked either by the chain or the ram. The former method is more usual with electrical, the latter with hydraulic, power.

The cloughs and their electrical connections at Ymuiden Locks are illustrated in figs. 528 to 530. Each frame (which is built of timber and sheet metal) is suspended by two endless chains fixed to the ends of a pivoted yoke at the top of the frame, and resting on a horizontal shaft above, through which they receive their motion before being carried round a secondary winding shaft half-way down the pit. The shaft is actuated by an electric motor situated in a separate chamber behind a partition

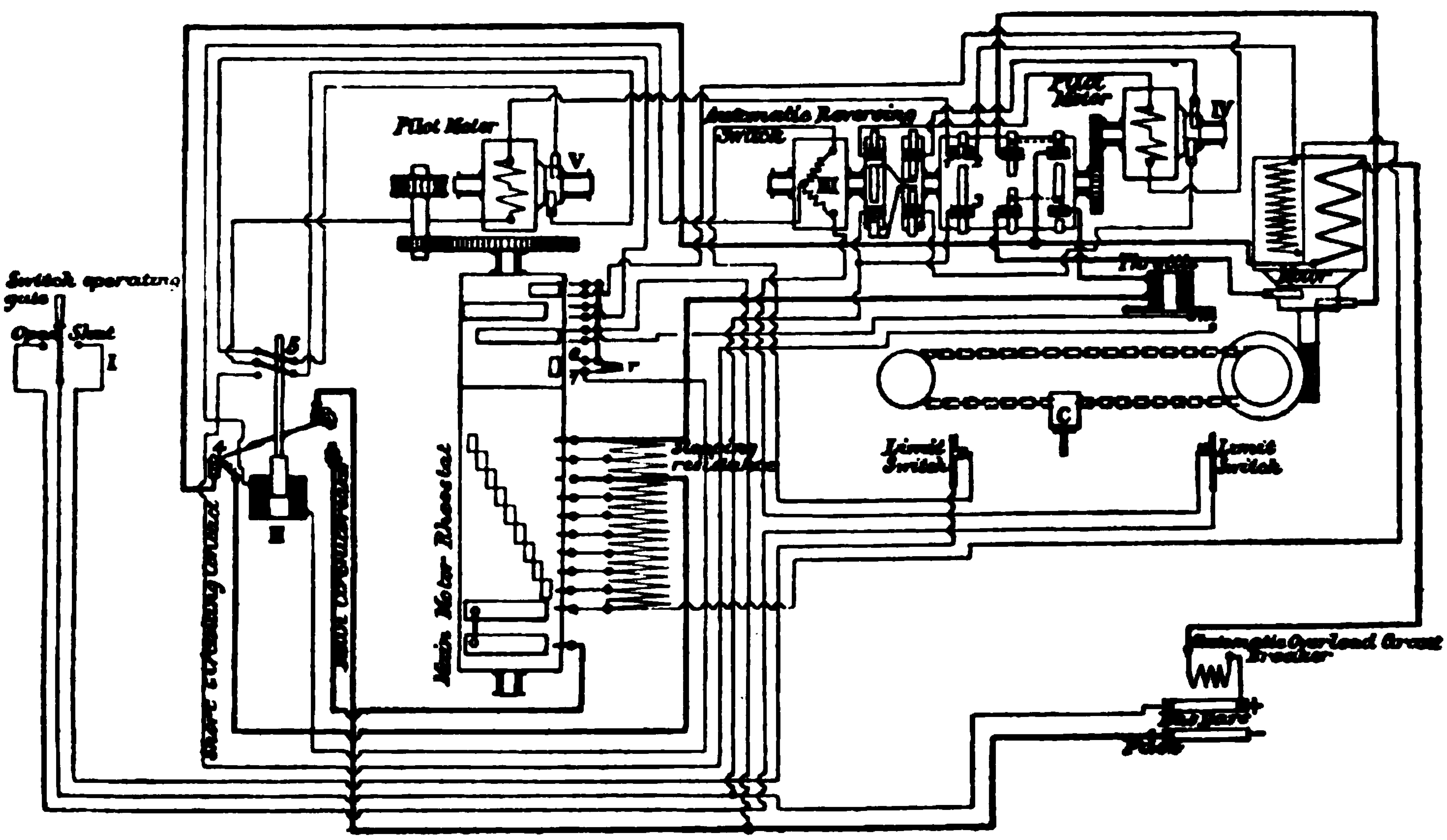


Fig. 530.—Diagram of Electric Connections to Gates and Sluices at Ymuiden Locks.

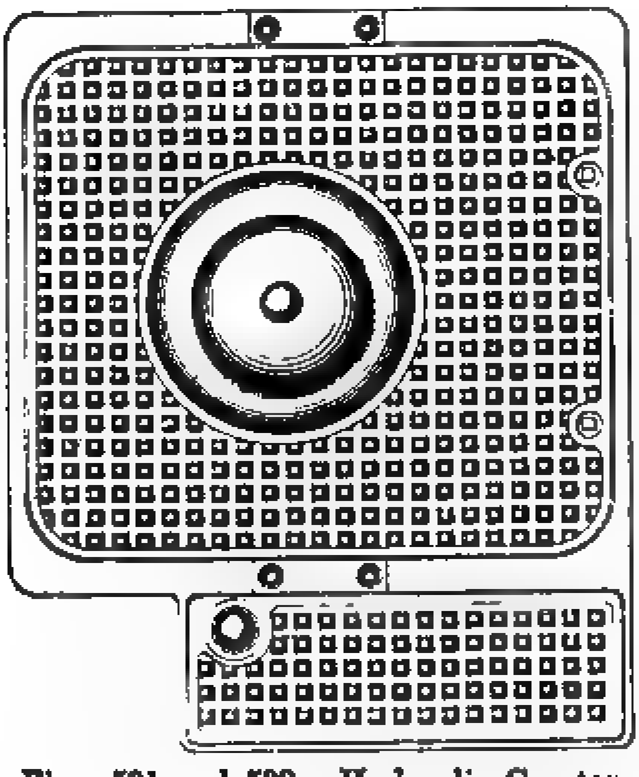
wall through which the shaft is carried. The motor is capable of developing 17 H.P. when running at 270 revolutions. The weight of the sluice is partly balanced by a counterweight, which is attached to the chain end and which glides on two rods provided with collars bearing against strong helical springs.*

In the hydraulic clough the frame terminates in a piston, which passes into a cylinder and is worked by differential pressure. Vertical guides are added to keep the frame in position during its ascent or descent.

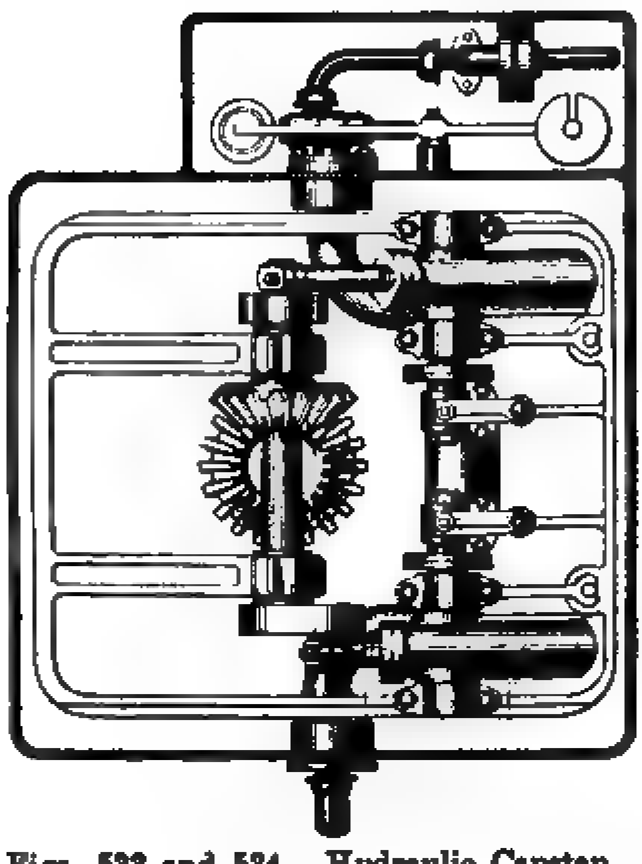
In case of failure of any part of the mechanical apparatus, it is advisable to provide a separate clough which can be worked by manual power. This is usually effected by a cross-bar at the summit of a spindle, with screw thread, passing through a fixed bracket.

* Articles on "The Electrical Gear at the Ymuiden Locks" appeared in *Engineering*, Feb. 7, 1902, and subsequent issues.

Power of Sluice Machines.—In estimating the power required for working clough paddles, there are two factors to be taken into consideration—viz., (1) the weight of the paddle itself, and (2) the working friction against the faces of the clough jambs. This latter is greatest at starting and will diminish as the paddle rises. The maximum effect can be found by calculating the pressure against the face of the paddle, due to the initial head of water, and multiplying by a coefficient of friction.



Figs. 531 and 532.—Hydraulic Capstan.



Figs. 533 and 534.—Hydraulic Capstan.

The following are values for the latter when the surfaces in contact are wet or dry :—

	Wet.	Dry.
Wood on wood,	·68	·50
,, ,, stone,	·70	·60
,, ,, metal,	·65	·60
Metal ,, ,,	·15	·18

Capstans.—Capstans belong to the same class of appliances as winches, the only difference being that their axes are vertical instead of horizontal. This arrangement favours the working of them by hand when necessary. Accordingly the capstan head should be designed at a convenient height and apertures for poles arranged in it, so that, in case of any breakdown in the usual motive power, the machinery may be actuated by hand. A pawl and ratchet gear along the lower circumference will prevent backslip.

Capstans of from 3 to 12 tons power are generally found sufficient for dock work. Excessive power would only result in the fracture of cables.

One capstan, at least, should be located at each side of an entrance, and if there be a long lock, two or four others will certainly be advisable at equal intervals. The position of a capstan should be such that, if there be a pair of gates in the vicinity, a convenient lead may be obtained for opening or closing the gates in the event of an accident to the gate machinery.

VERTICAL SECTION

Fig. 535.

Fig. 536.—Electric Capstan.

Capstans are obviously most, if not solely, adapted for working by means of rotary engines. In the case of hydraulic power, an illustration of the mechanism as devised by Lord Armstrong for a two-cylinder machine is afforded in figs. 531 to 534. The method of admitting the pressure water to

alternate faces of the piston-ram will be perceived from an inspection of fig. 535, which is a section showing the valve of the cylinder. A is the supply passage; B, the constant pressure port, always open to the upper side of the piston; C, the pressure port to the under side of the piston; D, the exhaust therefrom; and E, the discharge passage from the engine. F is a ring of hard metal forming the fixed working face, the upper segment of which, marked G G, is free to press up against the rubbing surface as it wears down, and is kept in contact by the pressure of the water. H is the trunnion in section, showing the pressure port on the upper side and the exhaust port on the lower; and I is the relief valve, the port to which is always open at the moment when the relief valve is required to act.

A vertical section of an electrically worked capstan at the Ymuiden Locks is given in fig. 536.

Quay Cranes.—Quay cranes are of all capacities, from half a ton or less to 150 tons or more.

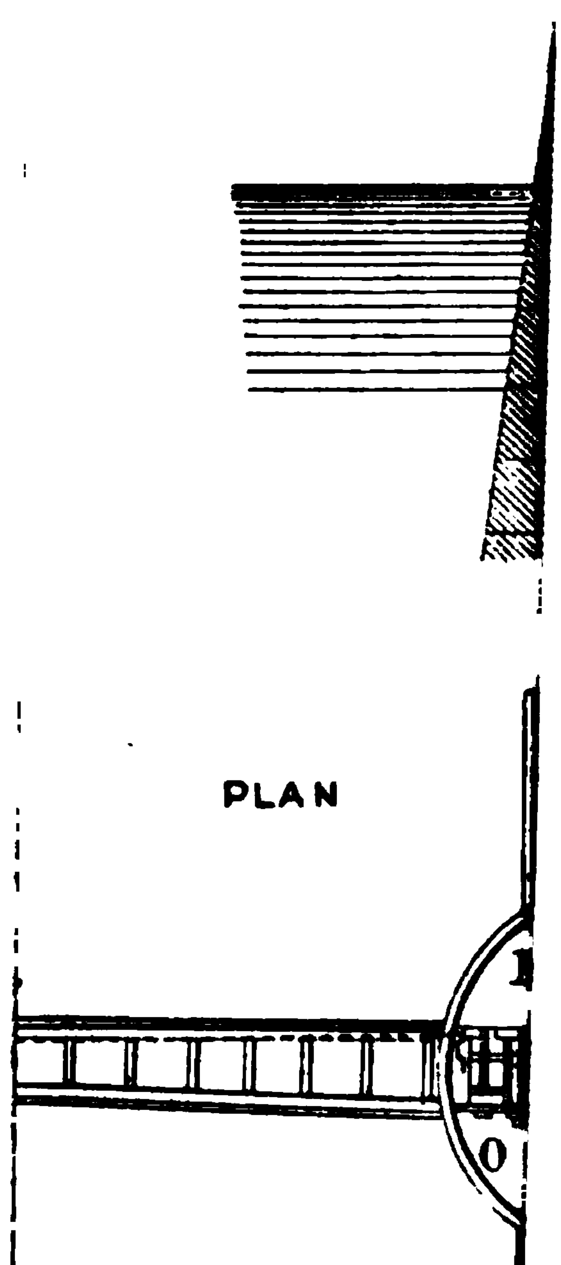
Types are innumerable, and it is quite beyond the province of this work to attempt to deal with them except on very restricted lines. For dock work, cranes may be concisely divided into two classes—viz., fixed cranes and movable cranes. The smaller class of cranes, dealing with the loading and unloading of vessels with cargo, are generally of the latter type, from the necessity of adapting them to the variable positions of the hatchways. They are subdivisible as follows:—

1. Cranes which travel upon rails all of which are at coping level. To accommodate the track, and also to ensure stability, this arrangement involves a clear space of some width—say, 10 feet—for the crane alone, and as additional tracks will generally be required for trucks, both while loading and in reserve, the width may easily be extended to 30 or even 50 feet. To reduce this large allowance, often inconvenient when space is limited, pedestal cranes have been devised, such that one or more lines of waggons can pass beneath the crane platform.

A hydraulic crane of the former type is shown in figs. 537 and 538 and a pedestal crane in figs. 539 to 541. In both cases the lifting is performed by a ram and cylinder with six sheaves.

The pedestal crane is the copy of one in vogue at Havre, Dunkirk, Bordeaux, and other French ports. It is adapted to two lifting capacities of 15 and 35 cwts. respectively. The different powers are obtained by concentric cylinders. A slewing motion is imparted by two hydraulic rams placed vertically behind the pivot. A single chain, common to both presses, is attached to the turning drum, so that the motion of one ram causes it to revolve in one direction while the motion of the other ram produces revolution in the other direction.

2. Cranes (fig. 393) which travel upon one rail at the coping level and upon another carried by a balcony or corbel on a transit shed at some height above the quay, generally at first floor level. This is obviously a device for



Figs. 539, 540, and

Fig. 542.—150-Ton Crane at Bremerhaven.

Fig. 543.—150-Ton Revolving Crane at Kiel.

Fig. 544.—100-Ton Derrick Crane at Hamburg.

Fig. 545.—100-Ton Revolving Crane at Bremen.

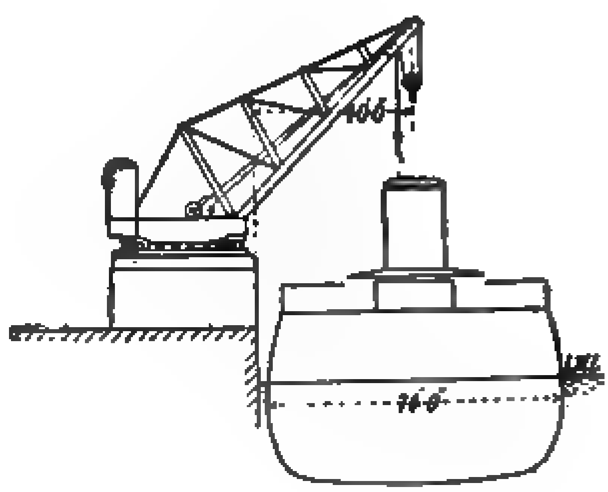


Fig. 546.—120-Ton Crane at Barrow.

gaining space on narrow quays. It embodies all the features of a pedestal crane minus the back legs.

3. Cranes (fig. 374) which are carried entirely upon the shed structure, either at some floor level or upon the roof. This arrangement is inevitable when there is not sufficient space to accommodate the crane upon the quay, and, in other cases, it avoids the obstruction caused by the front legs of the semi-pedestal crane, but it involves a corresponding increase in the amount of outreach.

The hydraulic roof-crane at Liverpool, shown in fig. 371, has luffing gear capable of altering the outreach or rake from 18 to 33 feet beyond the line of coping. The total height of lift is 76 feet, and the rate of lifting the full load of 30 cwts. is 150 feet per minute.

Fixed cranes have the advantage of greater stability, and are employed for lifting heavy loads. One at Malta, capable of raising a weight of 160 tons, is described at p. 536, *post*. Others, of various types, are illustrated in figs. 542 to 546.

The difficulty of employing large cranes with long outreach is the revolution of the jib amid the intricacies of masts, yards, stays, &c., of shipping. In many cases a pair of sheer legs, or oscillating derrick crane, is to be preferred. In form, the apparatus is a tripod with two legs pivoted horizontally at the edge of the quay and the third adjustable to the amount of outward projection. The movement of the load is entirely in one plane, at right angles to the direction of the quay, by which arrangement any interference with objects on either side is avoided.

Floating Cranes.—A floating crane, or sheers, is a valuable adjunct to the equipment of a dock system, as apart from its availability for shipping

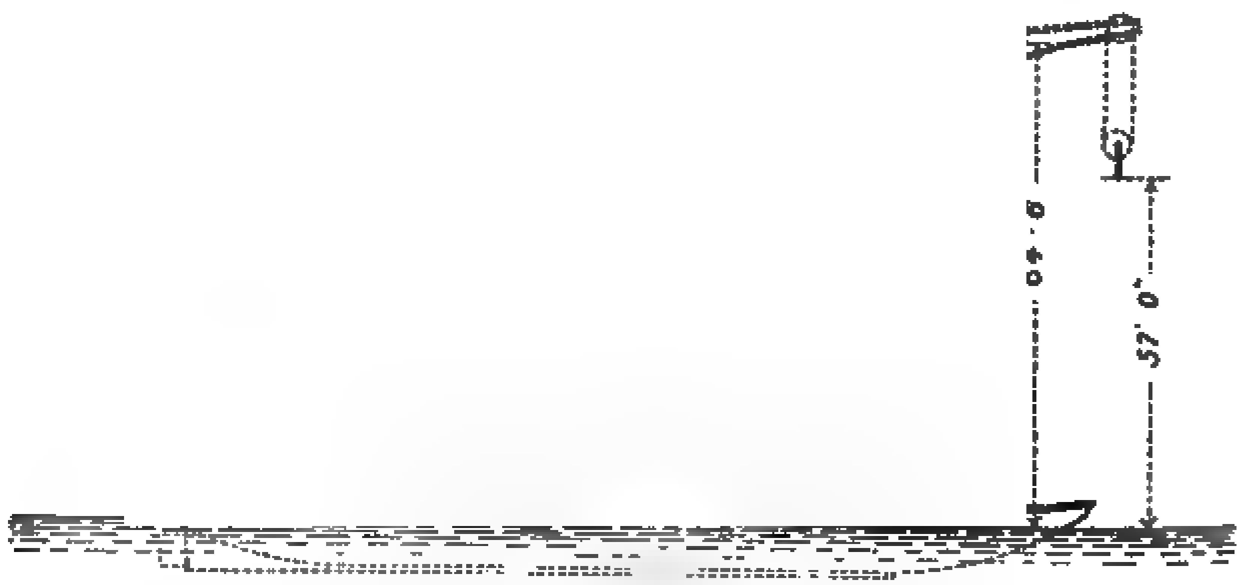


Fig. 547.—Floating Crane. Elevation.

and commercial purposes, it is of great utility in lifting dock gates for repairs, in berthing temporary dams, and in many other cases. Such cranes are constructed up to 100 tons lifting power. One of 25 tons is shown in figs. 547 and 548.

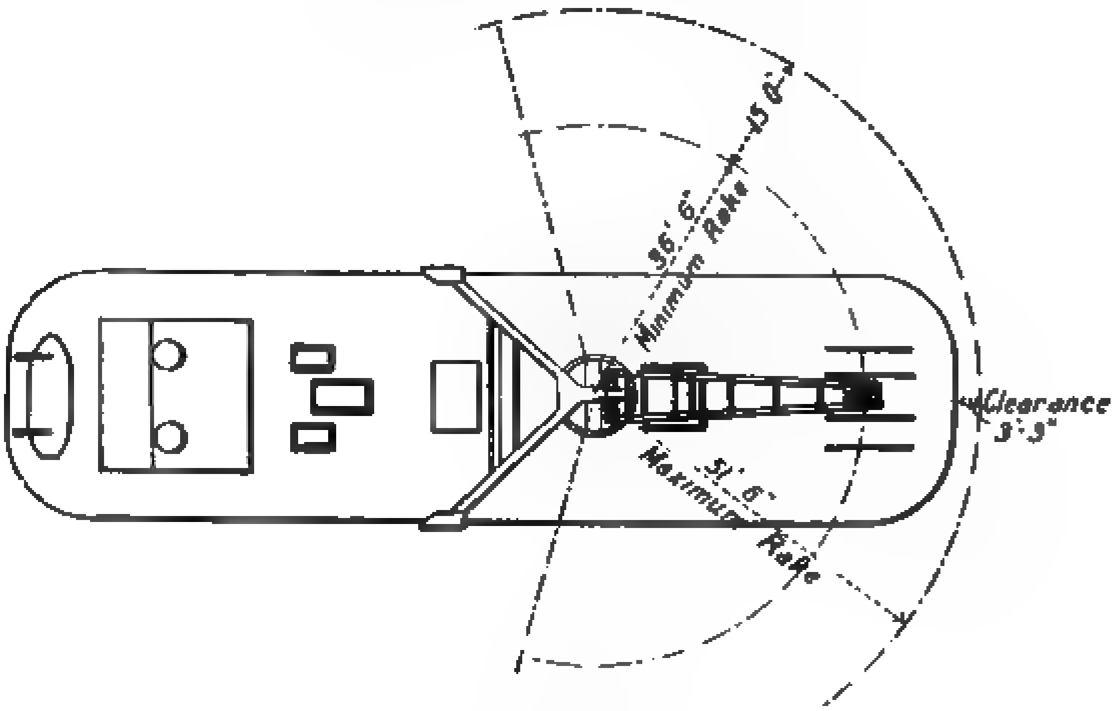


Fig. 548.—Floating Crane. Plan.

Figs. 549 and 550.—Hydraulic Jigger.

Jiggers.—Closely akin to cranes are jiggers (figs. 549 and 550) actuated commonly by hydraulic power. The apparatus is so light as to not require a rail track. There is no jib, and goods are simply hoisted out of a ship's hold by means of a chain, or rope, passing over a sheave suspended to the rigging. It may be used as a useful auxiliary to quay cranes, and it has certain advantages in rapidly lifting light articles out of the holds. It has a close competitor in this respect in the winches with which steamships are usually furnished.

Hydraulic Crane at Malta.*— This crane, the elevation of which is shown in fig. 551, has a maximum working load of 160 tons. This weight can be lifted through a height of 50 feet at a radius of 70 feet. Loads up to 35 tons can be lifted through a height of 90 feet at a radius of 75 feet. The larger loads are raised by means of a direct-acting hydraulic cylinder suspended in gymbals from the jib; the smaller loads by a chain purchase, worked by a rotary hydraulic engine.

The structure, which was constructed and erected by Messrs. Sir W. G. Armstrong & Co., of Newcastle, is carried by and revolves upon 96 bevelled live rollers, 15 inches mean diameter and 16 inches wide, working on a lower roller path of cast iron, planed on the top and bottom, the top being bevelled to suit the rollers. The rollers are connected on the outside by links, 5 inches by $\frac{3}{4}$-inch, passing over the ends of the axles, each link taking two rollers; and on the inside, the end of the axle is clipped in a wrought-iron circular frame, connected by bracing (figs. 552 and 553) to a collar, working on rollers round the centre column of masonry. The centre pivot is fixed to the masonry by four bolts, 15 feet long, extending into the centre chamber, and is made hollow to admit the hydraulic pipes. These pipes are concentric through the pivot, the internal one being for pressure and the other for exhaust. The foundation consists of a solid mass of Portland cement concrete, composed of 6 parts of hard limestone and 3 of sand to 1 of cement, faced above ground with limestone masonry.

The main lifting cylinder is of cast iron, in three lengths, connected by fourteen $3\frac{1}{4}$-inch bolts. Its internal diameter is 29 inches, and the thickness of the metal $3\frac{1}{4}$ inches. The piston is of cast iron and arranged for hemp packing. The piston-rod is of wrought iron, 8 inches in diameter, and fitted at the lower end with swivel eye and shackle. A platform is suspended from the cylinder, from which the inlet and outlet valves are controlled; it is reached by a light iron bridge hinged to it, and resting upon the framework of the jib. The cylinder is carried in a wrought-iron trunnion ring, suspended from the jib by four forged iron links, so that it can be swung in towards the jib when the 30-ton purchase is in use.

The rotary hydraulic engine for working the 30-ton purchase, the slewing machinery, and the swinging-in gear, has three oscillating cylinders,

* C. and C. H. Colson on "The 160-Ton Hydraulic Crane at Malta Dockyard Extension Works," *Min. Proc. Inst. C.E.*, vol. cxiv.

with gun-metal plungers, $3\frac{13}{16}$ inches in diameter and 14 inches stroke. All the levers for actuating the starting and stopping valves and gearing generally are arranged so as to be worked by one man standing upon a platform, raised above the lower framework of the crane. Turning motion is effected through spur and bevel gearing, acting on a toothed rack on the outer edge of the lower roller track.

All the machinery pipes and valves, subjected to a working pressure of 700 lbs. per square inch, were tested to 1,400 lbs., and the cylinder, piston-rod and piston, links, &c., were tested to 320 tons, or double the working load.

Transporters.—A transporter consists essentially of a long arm or track, placed horizontally or very nearly so, along which travels a carriage with a hook for the attachment of loads. There are several types of transporter; two will be briefly described.

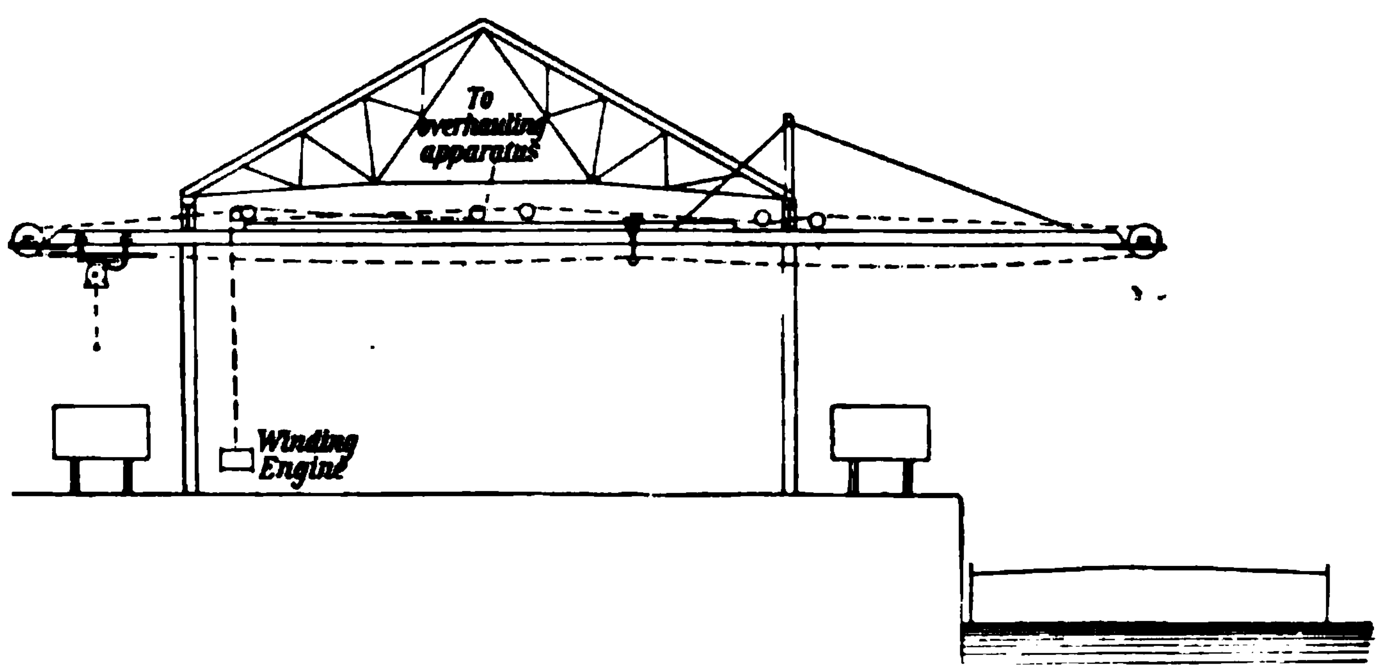


Fig. 554.—Temperley Transporter.

The *Temperley Transporter* consists of an iron beam of H section, supported by a special tower, by the mast of a vessel, or by the underside of a shed floor or roof. The traveller, or truck, is provided with an arrangement for throwing itself automatically out of gear at fixed positions vertically over the points of loading and discharge. It is actuated by a steel wire cable, which can be set in motion by steam, hydraulic, or electric power, and also by gas engines for lower speeds. These transporters are applicable to loads of from 5 to 60 cwts., and to distances up to 1,000 feet, with travelling speeds ranging from 500 to 1,500 feet per minute.

A view of the transporter is shown in fig. 554. The traveller, at the left end, contains automatic mechanism which secures it to the beam during the lifting of the load, and which sustains the load during the movement of the traveller. The load having been attached to the hook in its lowest position, the latter is hoisted by a hydraulic or other engine until the fall block enters the bell of the traveller, when two hooks automatically engage the block and sustain the load, while, at the same time, the traveller is released from the notch in the beam and commences to travel into the building.

These notches, which fix the stoppages of the traveller, are arranged at intervals of about 5 feet. On arriving at the position at which it is desired to lower the load, the engine is stopped, the hoisting drum thrown out of gear, and the traveller with its load commences to run backward under the action of a tail rope or overhauling gear until it comes to a notch, in which it engages automatically, and, at the same time, releases the fall block, so that the load can be lowered with the brake in the usual manner.

The outer projecting ends of the beams may be hinged, so that they can be drawn into a vertical position when out of use.

The *Transporter* shown in figs. 555 and 556 is of a type used in France, and manufactured by Messrs. Daydé and Pillé, of Creil. It is formed of a vertical framework in the shape of the letter A, at right angles to which the transporting beam is set. The frame is free to travel along the quay on a line of rails, and is steadied by a second line of rails placed along the shed front, at some distance above the quay level, so as to prevent overturning.

The travelling beam can be set to any required inclination. The apparatus is worked by two independent winches, one of which controls the hoisting of the load and the other the travelling movement. Both these winches are driven by steam power from machinery at the foot of the frame, movement being communicated by means of shafts and bevelled wheels.

The apparatus in question is adapted to loads of 30 cwts.

Coal Tips and Lifts.—The process of loading a vessel with a cargo of coal is attended by some difficulty, owing to the brittleness of the material, which is such that, unless extreme care be taken, its value may be very seriously depreciated by breakage into minute fragments and dust.

Coal is usually conveyed to port from the collieries in waggons, either end-tipping or with drop-bottoms. Waggons of the latter class are simply lifted and slewed bodily by a quay crane, and suspended over the hold while the coal is discharged. End-tipping waggons are tilted so that their contents are emptied into a shoot which directs them into the hatchway. Until a conical heap of sufficient height is formed, the operations in both cases are accompanied by considerable breakage of coal, owing to the great depth of the hold into which it has to fall. This can, to some extent, be remedied by the assistance of an anti-breakage crane, which forms an auxiliary feature of most coaling tips. For the first few waggon loads, the coal, after passing down the shoot, enters a skip placed to receive it, the skip being suspended from the crane, by which it is lowered carefully to the bottom and its contents there deposited. Even after the cone has attained a good height, it will be necessary to control the discharge from the shoot with the aid of flaps, or doors, as a rapid rush of material will frequently produce nearly as much damage as a long fall.

If the railway tracks are at a sufficiently high level, the waggons may be discharged direct from that level, but, in the case of a low-level approach, it will be necessary to first lift the waggons to such a height as will clear

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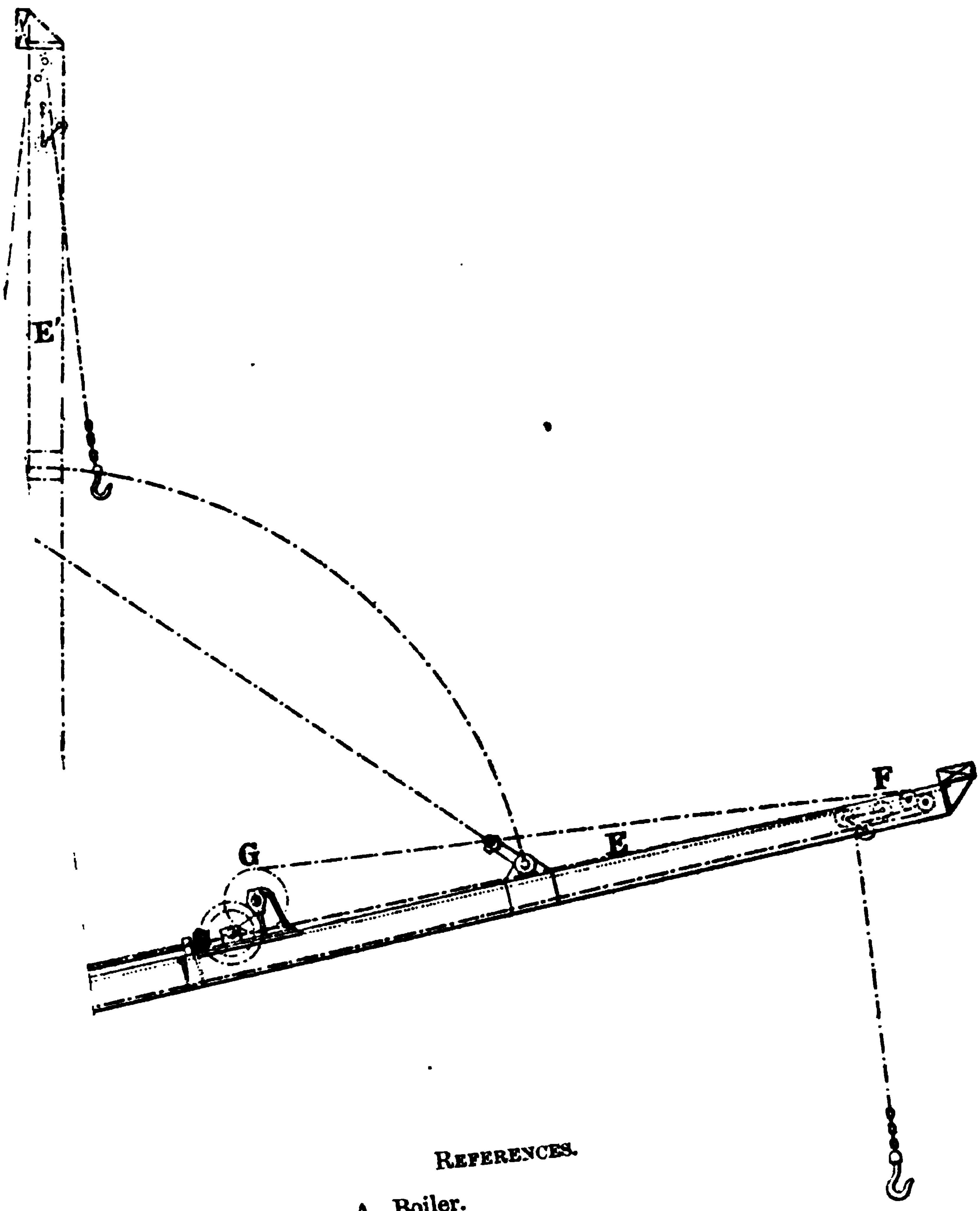


REFERENCES.

A, Boiler.
B, Steam Engine.
C, Speed Regulator.
D, Transmission Shaft.
E, Oscillating Beam.
F, Traveller.
G, Hoisting Gear.
H, Travelling Gear.
I, Upper Track.
K, Lower Track.
L, Traversing Gear.
M, N, Lifting Gear for Beam

the bulwarks and hatchway coamings, and, at the same time, give the requisite inclination to the tip.

Figs. 557 to 559 are illustrations of a hydraulic coal hoist and tip recently constructed at Dundee.*

The hoist is designed to lift a 20-ton waggon through a height of 50 feet above the level of the jetty rails, and at the summit to tip it through an angle of 45 degrees. Owing to the difficulty of providing suitable foundations at a moderate cost, the structure having to stand in the river 120 feet beyond the line of quay, a suspended form of hoist has been adopted, instead of that in which the cradle is raised by direct-acting cylinders placed in a well below the surface of the quay. The hoist framing is of steel, braced and strutted, and securely bolted to the timber-work of the jetty. The cradle and tipping frame are lifted and lowered by four chains, two of which are for lifting and two for tipping. The lifting cylinder is fixed vertically against one side of the framing, and the tipping cylinder is fixed on the upper end of the lifting cylinder. Each cylinder is fitted with a plunger, multiplying sheaves, guide bars, &c. The hoist is also furnished with a 2½-ton anti-breakage crane, having a lift of 55 feet. The structure is said to be the largest of its type.

Owing to the dust arising from the shipment of coal, it is essential to locate tips at a safe distance from quays for the reception of cargo of a nature likely to be affected by it.

Grain Elevators.—Appliances for dealing with cargoes of grain in bulk are necessarily very different from those employed in lifting packages and portable objects generally. In the case of a granular substance it is clearly advantageous to provide some method of uninterrupted transmission, such as that afforded horizontally by endless bands in revolution, and vertically by a succession of buckets on a continuous chain. Pneumatic power, in the form of either suction or pressure through tubes, can also be employed to achieve the same result. When the quantity dealt with is small, or when it forms part of a miscellaneous cargo, intermittent discharge by means of grabs, worked by cranes, may suffice.

The bucket system is in vogue at Liverpool and other places, and the rate of travelling reaches 100 feet per minute. An important drawback of the system is the limited range of self-feed for the buckets. They are only able to deal directly with the grain in the immediate vicinity of the hatchway. That portion of it which lies under cover, it may be to the extent of a hundred feet or more, fore or aft, has to be trimmed in the direction of the buckets, generally by manual labour.

The pneumatic system adopted at the Millwall Docks, London, and elsewhere, whilst entailing a greater consumption of coal than the bucket system, offers some advantages in other directions. The pneumatic tubes, being flexible, can be applied in any required position, and the cost of trimming is thereby saved, though at the same time the shifting of the

* Buchanan on "The Port of Dundee," *Min. Proc. Inst. C.E.*, vol. cxlix.

tubes necessitates attention, but not to the same extent. No matter how tortuous the route, the grain can be sucked out of bunkers and other confined spaces, which would be otherwise inaccessible. Furthermore, there is much less exposure to the weather, and pneumatic elevators can be worked under almost any atmospheric conditions.

Figs. 560 and 561.—Pneumatic Grain Apparatus.

The Millwall apparatus,* illustrated in figs. 560 and 561, is located in a hull about 80 feet long by 24 feet wide by 10 feet deep. It is driven by a compound engine connected direct with air-exhausting pumps, capable of producing and maintaining a partial vacuum of 15 inches of mercury, in a tank into which some 5,000 feet of air, under atmospheric conditions, is being admitted per minute. The tank, which acts as a grain-receiver, is

* Duckham on "Pneumatic Machinery for Loading and Discharging Grain Cargoes," *The Engineer*, April 8, 1898.

supported from the deck by a tower, and has a diameter of 10 feet with a height of 16 feet. It is coned at the bottom, and furnished with connections for two or three pipes, through which the grain is drawn with the current of air from the hold of the ship. An automatic air-lock is attached, and through it the grain discharges itself into the hopper of the weighing machine, whence, after weighing, it is directed into a barge in bulk or is filled into sacks. This type of machine is also in use at Bemerhaven and Hamburg.

In a pneumatic apparatus employed at Limerick, the grain, instead of flowing away in bulk, finds its way through a second air-lock into a chamber below the deck into which air is forced at a pressure of from 6 to 8 lbs. per square inch. From this a pipe passes upwards, bends over the elevator's side, and is there connected, by a piece of flexible hose, with an underground pipe passing up into and along the roof of a warehouse. By means of outlets provided at convenient intervals the grain is discharged into the required bins.

Slipway Haulage.—As originally devised by the late Thomas Morton, the inventor of the slip dock, the machinery for hauling vessels up the ways consisted of spur gearing worked by manual power, horses, or the steam engine. Hydraulic apparatus was introduced about the year 1850, and has since existed through various stages of development in competition with a form of winding apparatus originated about the year 1879.

The hydraulic apparatus in its later form, as contrived by Messrs. Lightfoot and Thomson,* consists of three main hauling rams (figs. 562 and 563), connected by means of an upper crosshead with a single reversing ram under constant pressure, and by means of a lower crosshead with a double set of hauling links which extend nearly to the extremity of the ways, resting upon wings cast upon the centre rails and being guided thereby. The action is as follows:—By the admission of water to one or more of the main cylinders, according to the size of the vessel being dealt with, a forward stroke of 10 feet is made against the constant pressure of the reversing ram. The main cylinders are then opened to exhaust, and the backward stroke is made under the action of the reversing ram. There is a dual system of pawls on the cradle, so arranged that one of them engages in the rack of the permanent way at the end of each forward stroke, while the other engages in the joint plates of the hauling links at the completion of each backward stroke. During the backward stroke, therefore, the cradle remains stationary upon the ways, while the hauling links are passing downwards to take up a new position 10 feet behind the pawls in which they were previously engaged. With this system, no disconnection or removal of links, such as obtained in earlier types, is required. The return stroke is made much more rapidly than the forward stroke on account of the much smaller area of the ram.

* Lightfoot and Thompson on "Slipways for Ships," *Min. Proc. Inst. C.E.*, vol. lxxii.

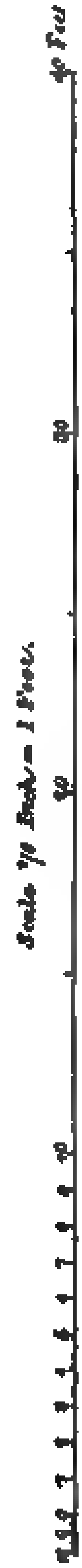


Fig. 562.—Hydraulic Rams for Slipway Haulage.

Scale, [illegible] inch = 1 foot.

Fig. 563.—Hydraulic Rams for Slipway Haulage.

A double set of cylinders and rams is the system adopted by Messrs. Hayward, Tyler & Co., and the apparatus is so arranged that one set is in forward motion while the second is returning. By attaching the links alternately to each set, the cradle is maintained in almost continuous motion.

The hauling gear of Messrs. Day and Summers consists of a wire rope, 12 inches in circumference, used either in single tension or with multiplying sheaves, coiled upon a drum, some 9 or 10 feet in diameter, which is actuated by steam or other convenient power.

The smoothness and regularity of the hydraulic ram commend it for the purpose of slipway haulage, particularly in dealing with vessels of large size. Steel wire rope, on the other hand, is light and flexible. Its durability has been contested, but appears to be satisfactory.

Pumping Machinery.—Permanent pumping power, as distinguished from that of a temporary nature, dealt with in a previous chapter, is required in connection with docks for two important objects:—(1) For emptying graving docks, and (2) for artificially raising the level of the water in wet docks. This latter expedient is adopted in cases where, greater draught being required for vessels, the deepening of a dock is deemed inadvisable on constructive or economical grounds. The use of pumping plant in connection with hydraulic accumulators is, of course, obviously necessary where such power is adopted.

The type of pump most commonly employed for the first named objects is that known as the centrifugal, in which the rapid rotation of a series of blades or fans causes the water within the pump chamber to be whirled round and propelled in an upward direction. Valvular pumps are unsuitable for dealing with dock water, on account of the great quantity of refuse matter to be found in it; corks, straw, chips, and ship scrapings are a few examples only of the multitudinous small objects which suffice to obstruct the action of valves. Centrifugal pumps themselves have to be protected by entrance gratings from the risk of entanglement with ropes and canvas, to say nothing of more serious damage by log-ends, pieces of planking, and wedges. It is no uncommon experience for a pump to have its intake pipe choked by eels and small fish, and the writer knows of one instance in which the pump blades were smashed by a piece of timber which had mysteriously intruded itself into the well. The following incident, narrated by Mr. John Hayes, is likewise instructive:—

Two large centrifugal pumps and engines, at Demerara, had been fitted up and set to work in connection with drainage operations on a somewhat extensive scale. One day, after they had been some considerable time in operation, the Resident Engineer observed that the engine and pump suddenly pulled up and then went on again immediately afterwards. For a long time the cause was undiscovered, but eventually the remains of an alligator, 14 feet long, were found in the outlet of the pump. The reptile had passed through the pump, and had been cut into three pieces, which the Resident Engineer caused to be stuffed, as a specimen of what centrifugal

pumps would do in the way of getting rid of obstructive *débris*. The alligator was undamaged except where it had been severed.*

Centrifugal pumps are of two types—the vertical and the horizontal. The latter is perhaps more generally known as the turbine. The turbine has an advantage over the centrifugal proper, in that the machinery for driving it can be placed at or about the quay level, whereas the other has its motive power applied near the middle of its lift, about half of which is done by suction, and the other half by propulsion. This involves an expensive watertight chamber below the level of the surface of the dock. On the other hand, the centrifugal pump is simpler in construction, being driven by the main shaft direct, while the turbine pump necessitates the interposition of gearing. The maintenance of a centrifugal pump is therefore less expensive, and on this ground it commends itself to the favour of engineers.

It is not proposed to enter here into details of pumping machinery. The subject is so extensive as to call for separate and specialised treatment, which may be found elsewhere. Some brief particulars relating to installations at several graving docks are given in Chap. xi.

Petroleum Storage.—Petroleum is imported into this country either in barrels or in bulk—the latter by means of specially constructed tank steamers. The barrel system is the less economical of the two, owing to the depreciation in the value of the imported barrels, which may amount to as much as 20 or 30 per cent.

An ordinary barrel is some 33 inches long and 25 inches middle diameter; it weighs about 64 lbs. when empty, 400 lbs. when full, and contains 42 imperial gallons. Barrels can be most conveniently and effectively landed or shipped by means of parallel tracks of angle iron, set up on trestles, where necessary, to give the requisite inclination. It is found that there is no disposition on the part of the barrels to leave the tracks, however great the speed.

Petroleum in bulk from a tank steamer is usually pumped through conduit pipes into a storage tank or tanks ashore. These tanks are cylindrical in form, built of plates of wrought iron, or steel, and suitably stiffened. A settling tank of similar construction is often included in the equipment.

The following particulars relate to the petroleum storage depôt at Barrow Docks :—

There are two installations. The smaller consists of two tanks, with a capacity of 2,500 tons each. In the other installation there are six large tanks, two small tanks, and a settling tank, with a total capacity of 16,360 tons.

The tanks are of wrought iron, cylindrical in shape, 64 feet in diameter and 33 feet high, with flat bottoms and low-pitched conical roofs of iron plates, supported by iron principals resting on an angle-iron ring, 2 feet

* *Min. Proc. Inst. C.E.*, vol. xcii., p. 178.

below the top of the cylinder. There are two other angle-iron rings, one at the top and the other at the bottom of the cylinder, and between these three rings of tee-iron.

The roof-plating is about $\frac{1}{8}$ inch thick, and the side-plating ranges from $\frac{1}{4}$ inch thick at the top to $\frac{7}{16}$ inch at the bottom. The tanks are set on a bed of sand and stand their full height above the ground.

A vent-hole is provided at the apex of the roof, with a screw-down cover, and there are manholes, with covers bolted on, in the roof and also in the bottom side-plates.

The wrought-iron settling tank is 36 feet in diameter and 5 feet deep. It is open at the top, and contained within a brick house octagonal in plan.

There are also large barrelling sheds and a cooperage.

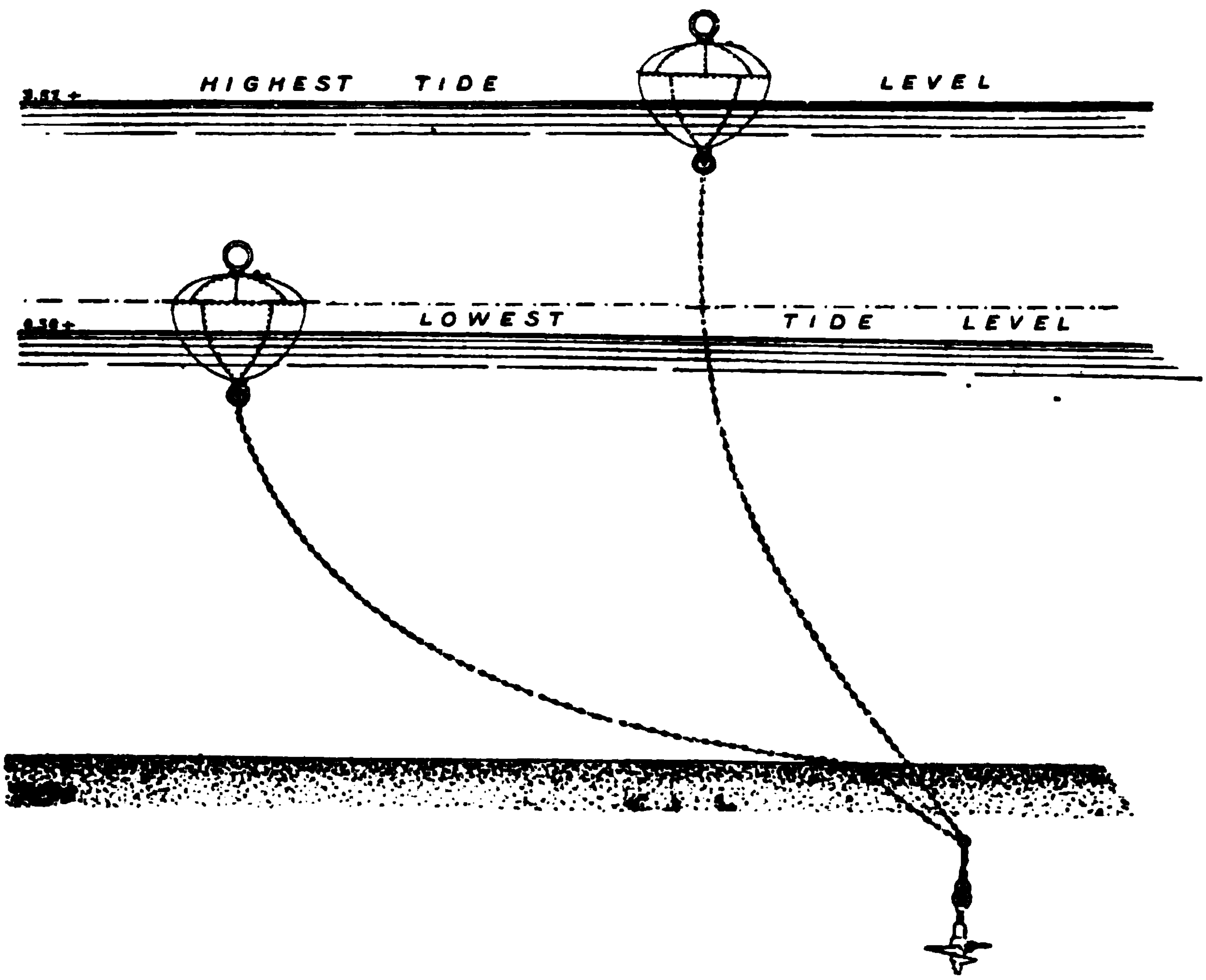


Fig. 564.—Buoy with Anchorage.

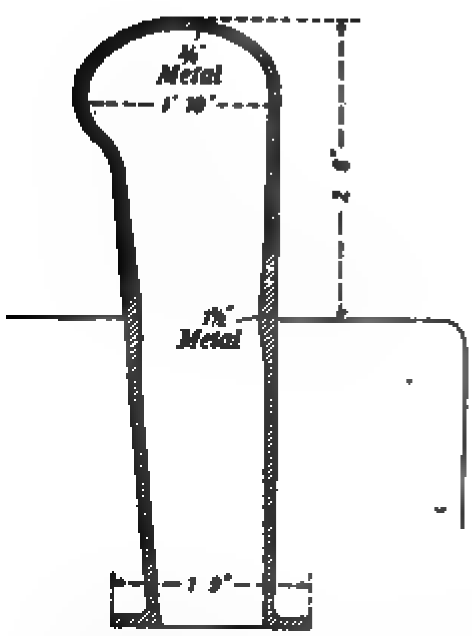
Ordinary Russian petroleum weighs $8\frac{1}{4}$ lbs. per gallon, American petroleum 8 lbs. per gallon. Petroleum increases in bulk 1 in 200 with an increase in temperature of 10° F.

Moorings may be classified as water moorings and quay moorings.

The former class, the object of which is to afford means of berthing ships while discharging cargoes into lighters in mid-stream or in creeks, includes anchored buoys and piled stagings. The buoys (fig. 564) are secured by chains to screw piles or to heavy blocks of masonry bedded in the ground. The stagings (figs. 565 and 566) consist of clusters of piles suitably braced and stiffened.

Quay moorings include rings, hooks, bollards, and posts. Rings and hooks, if placed in the vertical face of the quay, should be recessed so as to

Figs. 565 and 566.—Mooring Stagings.



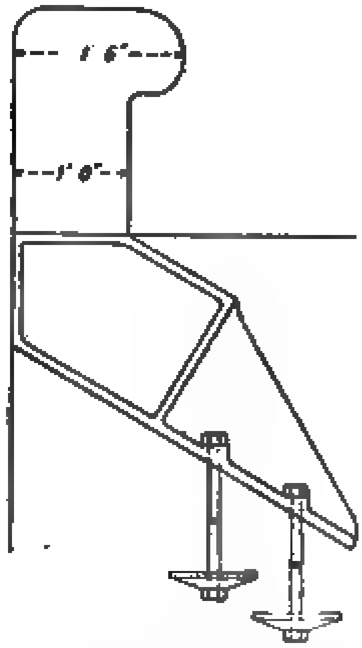


Fig. 567.—Mooring Post. Fig. 568.—Mooring Post. Fig. 569.—Mushroom.

avoid receiving or causing damage. Placed upon the quay surface, they are convenient for dealing with ships whose sides rise to a considerable height

above the quay. Hooked mooring posts and bollards, however, are the more general and satisfactory arrangement. These posts (figs. 567 and 568) are either of cast iron or steel, and, occasionally, of stone or wood. Hollow castings are undoubtedly the best, being strongest, most durable and compact, and comparatively light.

Mushrooms (fig. 569) are small, horizontal, single sheaves, placed so as to act as convenient leads for ropes and warps to capstans.

Dock Appliances at Hamburg.

The following is a list of various lifting apparatus in use at the port of Hamburg in November, 1901 :—

A. *Quay Cranes* (all fixed)—

1 steam and hand crane,	12,500	kilogrammes.*
1 electrical crane,	30,000	„
1 steam crane,	50,000	„
1 „	150,000	„

B. *Shed Cranes* (some fixed and some movable)—

278 steam cranes,	1,500 to 2,500	kilogrammes.
155 hand cranes,	1,000 to 2,000	„
101 electrical cranes,	2,500 to 3,000	„
27 hydraulic cranes,	2,000	„

C. *Miscellaneous Appliances*—

73 hydraulic winches or jiggers,	750	kilogrammes.
36 hydraulic winches,	600	„
25 hand winches,	500	„
75 „	750	„
14 „	600	„
39 hydraulic lifts,	1,200	„
3 hand lifts,	500	„
2 steam winches,	1,000	„

Equipment of the Port of Havre.

The following is a list of the various appliances belonging to the respective authorities in November, 1901 :—

A. *The Docks-Warehouses Company*—

4 fixed hydraulic winches,	400	kilogrammes.
10 movable hydraulic winches,	400 to 900	„
1 „ electric winch,	150	„
8 „ „ winches,	500	„
1 fixed hand crane,	10,000	„
2 electric grain elevators.		
8 „ capstans.		

* A kilogramme is 2·205 lbs. avoir.

B. *The Chamber of Commerce*—

30 movable hydraulic cranes,	.	1,250 to 3,000 kilogrammes.
2 ,, ,, winches,	. .	200 ,,
2 ,, ,, ,,	.	750 and 1,000 ,,
5 ,, steam cranes,	. . .	1,500 ,,
25 ,, electric cranes,	. . . .	1,500 ,,
1 steam floating crane,	. . .	4 tonnes.*
1 ,, ,,	. . .	10 ,,
1 tripod shears,	. . . . .	120 ,,

C. *Private Companies*—

3 floating shears,	.	2 of 30 tonnes, 1 of 7 tonnes.
8 electric winches,	. . . .	400 kilogrammes.
6 floating steam cranes,	. . .	1,500 ,,
5 fixed hand cranes, respectively,		5, 10, 12, 15, 25 tonnes.
7 movable steam cranes,	. . .	1 to 2 ,,
2 fixed steam cranes,	. . . .	$1\frac{1}{2}$,,
4 ,, hand cranes,	. . . .	1 ,,

A more detailed statement of the appliances used in connection with the working of a single dock at Liverpool will be found on the following page. The Canada Dock is one of the most important on the Mersey Dock Estate, and it accommodates the largest vessels of the White Star and Cunard Lines. The shipping companies, however, themselves provide the major portion of the appliances for dealing with cargo. These, and various manual appliances provided by the Dock Board, are not included in the list.

* A tonne is 1,000 kilogrammes = 2,205 lbs.

Dock Equipment at Liverpool.

The following particulars relate to the machinery in use for working the Canada Dock and its adjuncts, at Liverpool, the motive power being hydraulic throughout.

Location.	Class of Machine.	Type of Machine.	No. of Machines.	No. of Rams or Pistons in each.	Diameter of Ram or Piston.	Stroke of Ram or Piston.		Speed of Ram or Piston per Minute.	
					Inches.	Feet	Ins.	Feet	Ins.
Entrance lock,	Gate machines,	Rams with chains and multiplying sheaves,	5	1 main,	17½	16	0	9	0
				1 overhauling,	6⅜	3	9	9	0
,,	,,	,, ,, ,,	6	1	14⅜	13	2	9	0
,,	,,	,, ,, ,,	1	1 main,	18½	14	8	9	0
				1 overhauling,	7¼	3	7	9	0
,,	Footbridge machine,	Pistons with rack and pinion gear,	2	3	5	1	0	15	0
,,	Capstans,	,, bevelled gear,	3	3	3⅛	0	9	60	0
,,	,,	,, ,,	2	3	2⅞	0	9	60	0
,,	,,	,, ,,	2	3	5¾	1	3	43	3
,,	Sluice paddles,	Direct-acting rams,	3	1	11½	6	4	9	0
,,	,,	,,	3	1	14	7	6	9	0
,,	,,	,,	2	1	5	5	0	8	0
Main dock and branches,	,,	,,	1	1	14	7	6	8	0
,, ,,	Shed wall cranes,	Rams with chains and multiplying sheaves,	6	2 slewing,	3	1	4	·12	0
				1 lifting,	6½	3	8	30	0
,, ,,	,, roof ,,	,, ,, ,,	9	2 slewing,	7	1	3	6	0
				1 lifting,	10¾	7	5	30	0
Graving dock,	Gate machines,	,, ,, ,,	2	1 main,	18	12	0	9	0
				1 overhauling,	6½	3	0	9	0
,,	,,	,, ,, ,,	2	1 main,	18	10	6	9	0
				1 overhauling,	6½	3	0	9	0
,,	Sluice paddles,	Direct-acting rams,	2	1	11½	8	0	12	0
,,	,,	,,	3	1	8	4	6	12	0
,,	Capstans,	Pistons with bevelled gear,	3	3	3⅛	0	9	60	0
,,	40-ton crane,	,, and gear,	1	2	4	1	0	80	0

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